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JOURNAL
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RESEARCH NEEDS IN SEDIMENT HYDRAULICS

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(Proc. Paper 953)

SYNOPSIS

Considerable progress has been made in the past decade in the field of sediment hydraulics, particularly concerning the basic mechanics of the phenomena of sediment transport, erosion, and deposition. It is found, however, that there are many areas in which the tools available to the engineer for solution of sediment problems are inadequate. The Bureau of Reclamation in its existing and planned Federal projects is constantly faced with problems involving sediment. These problems include sampling techniques and methods of computing sediment load, design of stable channels in earth materials, channel stability when the natural regime of an existing channel becomes upset, sediment deposits in reservoirs and how they affect storage loss, aggradation above structures and degradation below structures, and exclusion of sediment from diversions into irrigation and powerplant canals. In the solution of these problems all the known knowledge and developments of sediment transport, scour, and deposition are used. However, in almost every phase there is need for research and development, particularly in the applied sense to increase the reliability and range of application of various formulas, and to develop practical procedures that can be applied in the solution of the various problems.

This paper points out some of the problems in which the Bureau of Reclamation in its work has experienced the need for more research in the field of sediment hydraulics.

INTRODUCTION

Because of the wide scope of the subject matter available, it has been necessary to limit the discussion to applying basic research to the solution

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of certain sediment problems. There are undoubtedly other important sediment problems requiring applied research, but which could not be covered in this paper. Computation and design methods are only touched upon and details on these methods can be obtained from the referenced publications.

As the title implies, this paper points out areas in which improvement is needed and in which inconsistencies exist in the field of sediment hydraulics. Actually the science of sediment transport, deposition and design, has advanced to a point where it is as accurate as most other hydrologic fields, and perhaps more accurate than some.

Transport and Deposition in Channels

Suspended Sediment

In the determination of the suspended sediment load in streams we have progressed a long way in recent years with the development of more accurate sampling equipment, improved sampling techniques, and more applicable procedures for analyzing the data and computing sediment transport. Incorporated with these improvements is the increased basic knowledge of how sediment moves in channels.(1)* However, there is still room for improvement and simplification in all phases of determining the annual suspended sediment load of a stream. Those who have made actual sediment measurements in the field, especially during high flows, and who have been burdened with case upon case of pint milk bottles filled with sediment samples, are fully aware of the need for technique and equipment improvements.

Measurement

Standard samplers. Suspended sediment samplers currently in use by most Government agencies include the US D-43, US DH-48, US D-49, and US P-46. The first three are depth-integrating samplers only; the DH-48 is a wading or hand sampler, and the fourth is a point-integrating sampler that can be used for depth integration. The US D-49 is a later and revised model of the US D-43. The development of these and other suspended samplers is described in a publication by the Federal Inter-Agency River Basin Committee.(2)

In the determination of the total sediment transport of a stream it is usually necessary to combine measured data with empirical computations, except when it is possible to sample the total load. Part of the reason for this is the physical limitations of the available suspended sediment samplers (Figure 1). The distance between the sampling nozzle and the stream bottom is given for the various samplers as follows:

US P-46	0.49 foot
US D-43	0.43 foot
US D-49	0.33 foot
US DH-48	0.30 foot

The percentage of the total sediment measured, therefore, varies with the stream depth and with the sediment concentration curve. Also illustrated in Figure 1 is the problem that results when sampling in a stream with moving

* Numbers refer to references at the end of the paper.

sand dunes. Here it is desirable to stay further away from the bottom than the limit of the sampler permits, to avoid obtaining erroneous suspended sediment concentrations. Using Imhoff cones in the field, the sand concentrations were determined for the Republican River near Wauneta, Nebraska. It was found that for point samples near the bed, obtained with a DH-48 sampler with valve adaptor, concentrations were measured which varied five-fold depending upon the location of the sampler nozzle with respect to the sand dunes.

Photoelectric cell method. Engineers are continuously searching for and devising methods which will do the required sampling and necessary intermediate work in a less costly and speedier manner. Use of the sediment sampling equipment which has become more or less standardized in the United States requires considerable time and manpower in obtaining the samples, analyzing them, and preparing the data in a form that is directly applicable for computing suspended sediment load. As an example, in an intermittent sampling program it is estimated that to sample a stream of 200 to 300 cfs at one station and analyze the sediment samples, assembling the data in a form that is applicable to plotting one point on a sediment rating curve would take about 1-1/2 man-days, counting travel time.

G. Bradeau, Engineer, National Hydraulic Laboratory, Chateau France, described a turbidimeter(3) that by using two photoelectric cells with light sources mounted in an instrument having a tunnel through which the sediment-laden water could pass, readings on an indicator would give the concentration and size analysis of sediment passing through the tunnel. Although the instrument is in the development stage, Mr. Bradeau indicated it could be utilized to determine sediment sizes and concentrations over a wide range. The use of an instrument such as this would cut down very considerably the time and manpower required to obtain and analyze sediment samples. The need for research and improvement is evident here.

The St. Anthony Falls Hydraulic Laboratory in Minneapolis, Minnesota, under the sponsorship of the Federal Inter-Agency River Basin Committee, is continually improving present samplers and working on the development of new ones. Perhaps the samplers have been improved more rapidly than have our sampling techniques.

Equal-transit-rate. Suspended sediment sampling is usually carried out by sampling at a given number of verticals across a stream channel or at centroids of sections giving equal discharge. A recent improvement in sampling technique is the equal-transit-rate (E.T.R.) method which has been developed by the United States Geological Survey. This method results in a more realistic sampling of the sediment movement since it integrates the product of concentration and velocity. It also results in a reduction in laboratory costs. Briefly, the procedure is to obtain samples at equidistant verticals while lowering and raising the sampler always at the same rate. The rate of raising and lowering the sampler is set by the deepest part of the stream where it is necessary to move the sampler fast enough to get not more than a pint bottle full in the deepest vertical. The samples thus obtained are consolidated for analysis. The E.T.R. procedure may be refined to the point where the water discharge could be determined from the sediment sampling data. Theoretically, the water obtained in the sampling process is actually a measure of discharge if a coefficient for the nozzle can be combined with the speed of raising and lowering the sampler to give an accurate integration of the velocity distribution.

There are, of course, limitations to the applicability of the E.T.R. procedure, and these limitations should also be established. It is impractical to set down a definite sampling procedure that will fit all field conditions but guide-type information is valuable.

Point sampling. It is sometimes desirable or necessary to obtain point samples in measuring suspended sediment. Experience with existing point samplers has indicated that improved models are needed. The mechanical features of the US P-46,(4) for example, make it a somewhat delicate instrument and therefore susceptible to operational troubles when field personnel are not familiar with the operation. Some improvements have been made to the US P-46 sampler recently.

A point sampling adaptor has been used by the Bureau of Reclamation in some instances (Figure 2A).. This adaptor can be used on either the US DH-48 sampler or the heavier depth-integrating samplers. It fits over the sampling nozzle and has a spring attachment which opens and closes a flap at the entrance to the nozzle by pulling a line. Where the depths are shallow the amount of water entering the exhaust port is so small that it can be neglected. For greater depths a flap or plug valve is used on the exhaust port. Although some small errors are introduced by the initial inrush conditions and the slight interference with the velocity pattern around the nozzle, they are not considered serious enough to affect the quantitative answers significantly. This adaptor has not received any laboratory analysis to determine its effect on accuracy of sampling. It is a field-developed piece of equipment that fulfills the need for a dependable point-sampling device that can be readily adapted to a depth-integrating sampler where for some reason the P-46 sampler cannot be used.

Another type of point sampler recently developed by the United States Geological Survey is operated by an air mechanism. In this sampler a small compressed air chamber is attached to the sampler and a valve in the sample nozzle is actuated by this pressure. The sampler has been developed primarily for wading-type samples and is currently receiving field tests that thus far indicate good results.

Sampling near the bottom of a stream. To obtain sediment samples near the bed in streams having hard bottoms, a special sampler has been developed. This sampler was made by the Bureau of Reclamation to obtain samples near the bottom in a constricted section of Fivemile Creek, Wyoming. It consists of a plastic tube 1-3/4 inches inside diameter and 12 inches long with a flap gate fitted on each end as shown in Figure 2B. The standard sampler DH-48 would not allow a sample to be taken close enough to the bed and the nozzles were too small for the largest sediment.

Computation of suspended load. The length of sediment sampling records on most streams in the United States rarely exceeds a few years. It is necessary in many cases to determine the probable long-time sediment transport of a stream using the available small amount of data. Suspended sediment loads for stations sampled by the United States Geological Survey are published in Quality of Water Supply Papers by River Basins.

Practically all reclamation projects require space to be allocated for sediment storage in proposed reservoirs. Reservoirs now constructed by the Bureau of Reclamation usually have space for 100 years of sediment accumulation before any of the water storage allocations are encroached upon. In

providing such space, consideration is given to the cost of the additional investment over the period of years before the additional space is needed. Of course, many reservoirs on streams with relatively low sediment loads will have no appreciable storage loss for many hundreds of years.

Several methods are currently in use by various engineers and agencies to predict the long-time sediment flow of a stream. The Bureau of Reclamation uses the flow-duration, sediment-rating curve method.(5) This procedure, as well as most others, is affected by many factors such as the amount of sampling data available to define a sediment-rating curve, the seasonal variation of the data, and the flow-duration curve for the years of sampling which may be wet or dry years. An example of this method, including seasonal variation, is shown in Figures 3 to 5 for the San Juan River at Bluff, Utah.

Generally speaking, most of the sediment transported by a stream, in any given period, is carried by discharges that exceed the median, see Figure 5. Sampling programs could perhaps be modified to reduce the number of samples obtained during low flows and increase the number obtained during higher flows. Sediment studies consistently indicate that there is a narrow range of discharges in any stream which, if sampled, would give a measure of the average sediment load carried by that stream. These factors all point to the possibility of using selective sampling in arriving at the average sediment load. Examination and analysis of flow-duration and sediment-rating curves would lead to a definition of the range in discharges that, if sampled, would give the average annual sediment load for various stream types.

Pump sampling. Often the sediment concentration in a particular stream or channel is so small it cannot be accurately determined with the ordinary suspended sediment samplers because it is not possible to integrate the flow over a sufficient period of time. In most cases this low sediment concentration would not be important, but there are times when its determination both in quantity and size is important. An example is the diversion of water into a powerplant. One method of sampling this type of flow is to use a water pump connected to a pipe so arranged as to sample the flows and discharge into a container of known volume such as a 50-gallon oil drum. The velocity of the water at the intake is adjusted to stream velocity by means of a valve. In this way the flow can be sampled over a long period of time, the necessary information obtained to determine concentration, and sufficient material obtained to analyze for size and quantity. This procedure, however, is cumbersome, and a simpler procedure is desirable. Undoubtedly others have been and will be confronted with this sampling problem.

Size analysis of suspended sediments. One of the greatest costs in a sediment sampling program is the size analysis. Therefore, any sampling procedure that will result in fewer samples for laboratory analysis without decreasing data accuracy or increasing field costs would be desirable. Present procedures require from three to seven verticals with two samples at each vertical to arrive at one river station sediment concentration and size analysis. If a single sample could be obtained for a given station in a stream at a certain discharge that would give a representative answer, only one concentration and size distribution analysis would be necessary. The E.T.R. sampling procedure mentioned previously accomplishes this to some degree.

Reduction in size analysis costs could also be accomplished by improved laboratory techniques. The visual accumulation tube(6) method of size analysis for sands which is becoming more and more widely used in this country

is one result of searching for faster and less costly methods.

Accuracy of determining the size distribution in a given sample is of prim importance since this size distribution reflects directly into the methods of determining sediment load. Sediment transport determination by such procedures as those advocated by Einstein[7] and Schroeder and Raitt[8] depends upon the size distribution. The above points are covered in more detail elsewhere but are mentioned here to illustrate the need for accurate size analysi determinations.

Most sediment sampling programs are necessarily restrictive because of high sample analysis costs. A single size analysis costs approximately $15, and a single concentration analysis costs $2. Any method that can be developed to reduce size analysis costs would be advantageous.

Unit yield method of determining sediment transport. From the sediment sampling that has been carried out in various sections of the country and fror data obtained from the resurvey of existing reservoirs, the unit yield or sediment runoff per square mile of drainage area, with time, is available for many streams and in many locations. When it is necessary to make a reconnaissance-type estimate of the sediment transport at a particular location on which no data are available, the unit yield from a similar type of drainage basin is often used. It would therefore be helpful to all agencies involved in making sediment estimates if all available data on sediment yields by type of drainage basin were readily available. The Federal Inter-Agency River Basin Committee has accomplished much along this line by its recent publications "Summary of Reservoir Sedimentation Surveys Made in the United States, June 1950"[9] and "An Inventory on Published and Unpublished Sediment Load Data."[10] It is expected these reports will be kept up-to-date Most agencies have carried out sediment studies for many streams by analyzing sampling records, and availability of these data would be helpful. Althoug some disagreement is inevitable in certain instances, a compilation of unit yield data from these studies would be helpful to all agencies and would reduc duplication of effort. In some drainage basins a large percentage of the total sediment is contributed by a few tributaries that produce only a small portio of the basin water runoff. To obtain an overall picture of the sediment and water yield distribution for the same drainage areas, a basin balance is prepared which shows water and sediment for the various sub-basins as a percentage of the total basin runoff (Figure 6).

Bedload

Involved in the determination of unmeasured or bedload is the definition of channel hydraulics and bed material size distribution. In certain streams the unmeasured load is insignificant; in others it embraces a large percentage of the total sediment in transport. For example, on the Middle Loup River at Dunning, Nebraska, more than 50 percent of the total load is unmeasured whereas on the Milk River near Nashua, Montana, the unmeasured load is estimated at 5 to 10 percent of the total transport.

Sampling bed material. An important part of the field data needed in determining the bed material transport and total transport of a particular stream is the bed material size distribution. The importance of obtaining representative samples of this type should not be underestimated since most formulas for computing bed sediment transport require a determination of

bed material size distribution. An error in the mean sediment size can result in an erroneous answer on the unmeasured load. For example, in the Schoklitsch bedload formula(11) a change in mean bed size (50 percent finer) from 1.1 mm to 0.8 mm results in a 100 percent increase in the bedload transport, all other factors remaining the same.

There are many types of equipment available for obtaining underwater bed material samples in normal alluvial stream channels. However, there are certain phases of the procedure that are not yet adequately defined, for example:

a) How many samples are necessary to define the mean?
b) Is there a change in the mean bed size with discharge?
c) How is a representative sample obtained in a cobble-bedded stream?
d) How are representative samples obtained under deep flowing water?
e) How deep should a sample be taken to give particle size information applicable to the various bedload formulas.

Typical bed material size analysis curves for the Middle Rio Grande River at Section F in the Bernalillo Study reach are shown in Figure 7. Considerable variation in size across the river bed at this section can be noted. The studies being carried out by the Bureau of Reclamation in cooperation with the United States Geological Survey utilizing the data obtained in this study reach include the application of the Einstein bedload function and the Modified Einstein Procedure to an alluvial-type stream having a large change in the stage-discharge relationship.

The problem of obtaining a representative sample from the bed of a channel containing a preponderance of cobbles or gravel has become more prominent in recent years in this country as additional developments are planned and constructed in mountainous areas. The particular nature of some projects, such as diversions into powerplants, makes it necessary to remove the material in transport. Various methods are employed at present such as obtaining several sacks of material by use of a shovel, at low flow stream bank water surface level, or perhaps by a pickup method such as described by Wolman.(12)

Bedload samplers. Bedload problems have not been as numerous in the United States as those of suspended load because most of the important rivers have beds composed of fine material and therefore carry much more sediment in suspension than along the bed. Because of this, the development and use of bedload samplers in this country have not kept up with that of suspended sediment samplers. In the 1940 Report No. 2 "Equipment Used for Sampling Bed Load and Bed Material,"(13) various bedload samplers are described. Except for a few experimental cases the samplers discussed were developed in foreign countries and used there.

It is believed that a bedload sampler for sand bed material could be developed which would give a better indication of actual bed movement than is indicated by use of bedload formulas.

One of the difficulties in developing an adequate bedload sampler has been preventing scour around the apparatus and having the sampler conform to the sand waves of the stream bed. Possibly using one of the new tough pliable plastics for a bottom would work. For sands a comparatively small opening upstream with a Venturi-type pressure reducer to extract the water and leave the sediment in a reservoir of the sampler may work effectively. The

Venturi principle is used on some suspended load samplers to obtain the proper intake velocity. This same principle could be used on bedload samplers to obtain the proper intake velocity. Any bedload sampler could be used if calibrated for conditions similar to those in which it will be used. Obtaining a few bedload samples would give greater confidence in using bedload formulas that give comparable answers.

In streams where the bed material is very coarse, the question frequently arises: At what flow does the coarse material begin to move? Georges Labaye(14) describes the use of a hydrophonic detector in which the beginning of transport of coarse material was indicated by sound against an immersed plate and amplified by a microphone pickup. For wide streams this method could be used to determine the fraction of the total width over which the bedload was moving.

Computing bedload. In computing the sediment loads of streams for planning reservoirs, an estimate based on empirical formulas is usually made for the unmeasured load. Most bedload formulas are based on drag flume experiments, and when applied to river problems may not give accurate results. Different formulas give widely different bedloads for the same stream, as shown in Figure 8. This is probably due to the fact that each formula works well for the conditions under which it was developed, but these conditions were different for different formulas, and when applied to river problems they give widely varied answers. Each of the various formulas that have been developed requires essentially the same basic data; channel cross section, water temperature, water discharge, energy gradient, and size distribution of bed material. Some of these factors can be defined fairly accurately by field measurement, while others are not so easily obtained. Some recent procedures for determining the bed material load have been developed that do narrow down the deviation in answers obtained.(7)(15)(16)

It should be the ultimate goal to produce sediment records that evaluate the total transport of sediment and its size distribution for a particular river and section. Most suspended sampling is currently being carried out by the United States Geological Survey, often at their regular stream gaging stations, and records are published in Water Supply Papers. If the total sediment load was published or some ratio or percentage of measured to unmeasured load, it would be much more valuable to those concerned with using sediment records. The suspended sediment records are only a partial measure of the total transport.

Since there does seem to be a definite relationship between the coarser fractions of suspended sediment load and bedload for varying stream types, it might be possible to improve or elaborate on relationships such as developed by Lane and Borland(17) by evaluating existing data from streams in this country and throughout the world. This method of arriving at bedload transport is considered good only for reconnaissance-type estimates. It can, however, point out the relative magnitude of the unmeasured load and help determine the need for any further more detailed investigation.

Channel Stability

Cohesive materials and apparent cohesive materials. The reclamation engineer is continually faced with the problem of choosing proper design criteria in designing canals in earth materials. The problem is to select the

smallest cross section for economy but which is large enough to carry the design discharge on a slope that will not cause the earth material to scour. A method for design of canals in non-cohesive materials[18] has been developed in the Hydraulic Laboratory of the Bureau of Reclamation. It is based on the critical tractive force principle, i.e., a given size of sediment on a stream bed requires a certain tractive force τ = (wds) to move it. The use of this principle to design the shape of the canal[19] in conjunction with the slope is not difficult for noncohesive materials. However, for clays and other plastic soils where cohesion is a predominant force (greater than resistance due to weight of individual particles) this process cannot be used directly because the resistance to erosion due to cohesive forces has not been determined. There is possibly a soil mechanics test or group of tests that can be correlated with resistance to scour or critical tractive force. The plasticity index, which is the difference in percent of moisture between plastic limit and liquid limit in Atterberg soils tests,[20] has been suggested as a soil characteristic that can be used to indicate resistance to scour. For canal design it has been mentioned that a plasticity index (P.I.) of 7 is the critical value, with scour occurring for moderate tractive forces below this value. Cases frequently occur where the P.I. is above 7, but we still get scour. Some fine silty type soils such as volcanic ash and glacial flour may have an apparent cohesion and even show a P.I. higher than 7. However, if observed under water, their soil structure breaks down readily when subjected to a hydraulic tractive force.

This emphasizes the need for laboratory tests and field data correlating soils characteristics with critical tractive force and resistance to shear. Plasticity index in combination with consolidated shear tests may be characteristics that can be used.

Vegetation. In many waste channels and some canals it is desirable to increase the stability of the soil by planting or encouraging growth of vegetation. For slow velocities, grass causes considerable resistance to flow, but for high velocities, where grass is forced to lie down, the resistance to flow may even be decreased. Certainly on ephemeral drains grass would be desirable, but if a complete grass cover cannot be assured over the full length of the wasteway, scour will result from high velocities in the denuded areas.[21] It may be desirable to allow enough water to escape into a drain to assure a vegetative growth. Research in the field of channel stability should include an evaluation of the influence of vegetation on the resistance to erosive forces.

Channels carrying suspended sediment. There is a definite effect on channel stability where suspended sediment is carried in the stream, but just to what degree has not been determined on a scientific basis. Fortier and Scobey[22] observed the stability of several canals some of which carried clear water and others carried suspended sediment. Their table shows allowable or critical velocities for both conditions. Lane[18] plotted allowable tractive forces for clear water, water with low content of fine sediment, and for water with high content of fine sediment. The data of these researchers indicate a higher allowable tractive force for water-carrying suspended sediment. Fortier and Scobey say, for water-carrying colloids, there should be a relationship relating characteristics of bed sediment, suspended sediment and hydraulic characteristics of the stream that will show just how the allowable tractive force can be raised for a given concentration of suspended sediment. The concentration of suspended sediment very likely affects the

stability of cohesive soils as well as bedload movement and formation of dunes in noncohesive soils.

Ephemeral and perennial streams. The degree of stability of stream channels, such as proposed project drains, is dependent to a large extent on whether the channel is or will be ephemeral or perennial. For the purpose of this paper, an ephemeral stream is one in which the flow is discontinuous and is directly related to storm runoff in rivers and to irrigation return flow in drains and occurs for short periods of time, seldom exceeding a few days. A perennial stream is one in which the flow is continuous for long periods of time each year, even though the stream may become dry at the end of the runoff season. It is known that an ephemeral channel can withstand higher tractive forces than a perennial channel. Since in ephemeral channels vegetation can develop and damaging high flows are of short duration, the bed and the banks do not become saturated and therefore do not deteriorate under the same tractive forces as a perennial channel. Vegetation, in most cases, cannot develop under constantly flowing water conditions, and this is one reason why perennial streams are not as resistant to forces of flowing water as ephemeral streams. It is necessary in the design of a surface drain to determine the probable nature of the flow and then design the channel so that the occurring tractive forces will not produce excessive channel deterioration.

The most striking examples of the effect of the flow duration pattern on channel stability are those in which flows introduced into a channel are different from the flow duration pattern for which the channel was developed and became stable. The new flow in a channel that has alluvial-type materials will cause rapid deterioration as the channel attempts to adjust to the new conditions—often resulting in expensive damage and costly repair.(23)

Research is therefore needed to define adequately the limiting tractive forces for various types of channel conditions. This research can be a combination of laboratory and field data. There are numerous channels on irrigation projects in this country where field data can be obtained for a wide variety of conditions. Some work has been done along this line, and additional studies are currently underway by the Bureau of Reclamation and others.

Stabilizing methods. The need for research in the field of channel stability is not only one of determining what flow characteristics and sediment load result in a stable channel but also what method of control should be applied in stabilizing channels already deteriorating and in maintaining the channel shape determined by some applicable method. Assuming that the channel shape has been adequately defined, the type of control installations to hold this shape must be selected. Many types of channel control are used. These include jacks (wooden, concrete, and steel), groins (brush or rock-filled), riprap, check dams, pervious fencing, and vegetation. The engineer must determine the applicable type of protection that will result in the most economical design, minimum initial expenditure, and low future maintenance cost. It has been found that a single row of jacks is most effective for flows in which they are almost completely inundated and on streams that carry considerable debris and fine sediment. Wooden jacks or a more permanent type can be used. If wooden jacks are used it must be assumed that their life is only between 5 and 10 years. If they deteriorate, the question arises as to whether planted and/or natural vegetation will hold the shape created by the jacks. Usually vegetation is employed in conjunction with jack installations so the job of holding the channel will fall upon the vegetation that has

developed. This being the case, it is necessary to evaluate the ability of the vegetation to resist erosion and the conditions under which it can no longer withstand the erosive forces.

Groins are found to be most effective in protecting banks at lower flows or flows at which they are not inundated. The correct spacing and the angle to the flow are all-important. Often groins and jacks are used in combination. If rock riprap is used the required thickness, the points in the channel layout where it is needed, and the quality of rock required must be established.

Riprap below structures. A special case of stable channel design is that of channels below hydraulic structures where turbulence and nonuniform velocity distribution occur. To take care of the extra forces which are created by drops, chutes, canal headworks, sluiceways, and other similar structures, stilling basins supplemented by riprap on earth slopes are provided. Choosing size and thickness of riprap and length of canal on which the riprap should be placed has been a matter of judgment and experience. Comparing scour patterns in coarse sand for model studies below spillways, stilling basins, canal drops, etc., has been used extensively to establish uniform distribution of erosive forces.

Canal beach slope. On practically all earth canals built of gravelly material that are operated at a constant depth a beach slope can be noticed at the water surface which is flatter than the side slope. At the water surface, the canal banks are subjected to waves caused by wind and other disturbances, and these waves exert forces which cause the earth material to wash down and take a flatter slope than the constructed side slope. On earth canals that are constructed with side slopes of 1-1/2 to 2:1, it is not uncommon to observe beaching slopes at the water surface of 5 or 6 to 1.

In conducting some wave studies in the Hydraulic Laboratory[24] to determine the required cover blanket to prevent fine base material from leaching due to wave action on Kennewick Main Canal, Yakima Project, Washington, several tests were made on the pit-run gravel shipped from the canal site. Figure 9 shows a beaching slope of 5.14 to 1 on the gravel material which had been subjected to 42 waves per minute with a wave height of 0.43-foot trough to crest for a period of 19 hours.

Canal design which could include a flatter slope in the water surface area would approach more closely the natural tendencies in canals constructed in gravelly materials. Because of this tendency to take a beach slope near the water surface, a design which included a flatter side slope near the water surface would be approaching more closely the natural tendency of the canal.

Gravel blanket over fine-grained noncohesive material. Many canals are built in areas where the earth material is fine-grained and noncohesive. The material is usually of rather uniform size which means once erosion begins it does not stop from the effect of larger particles being exposed. The canal is usually stabilized by artificial means. The use of a gravel blanket has proved quite satisfactory in this case. Especially where seepage is not a problem, the gravel blanket is very useful in providing resistance to higher critical tractive forces, resistance to freezing and thawing forces, and the cost is low compared to concrete and asphalt concrete.

A typical example of a canal under construction in fine-grained material is that of Kennewick Main Canal referred to under canal beach slope above. The base material has a median grain size of 0.05 mm with 20 percent

coarser than 0.12 mm and 5 percent finer than 0.005 mm. This base material has an apparent cohesion when compacted at the best moisture content, but when placed under water, the cohesion readily breaks down. Hydraulic Laboratory Report No. Hyd-381[24] describes wave studies made on the base material, a pit-run gravel, and an angular talus material to determine the best method of placing a protective blanket to give most resistance to leaching of the base material. The model studies showed that the angular talus material which had been separated on a 3/4-inch screen with the coarse fraction placed over the fine fraction gave the best protection to the base material.

The problem that is very important is how thick a gravel blanket to use and what restrictions must be made for the preparation and placing of the gravel blanket. For practical and economical reasons, a pit-run gravel offers the best solution, but from an operation and maintenance standpoint the pit-run gravel available may have a large preponderance of fines which will wash out, causing the coarser particles to settle or roll down the slope. The result may be a cover so thin it would not be adequate to give protection, and also, the canal prism may have a tendency to be filled up. The ideal gravel blanket is material that is placed so there are large enough particles on the surface to prevent movement from tractive forces and surface waves with enough sand between the coarse gravel and original bank material to prevent leaching of the bank material.

Methods of design of stable channels. The research needs in the design of stable canal sections in earth materials may be stated in this question, "What are the limiting tractive forces that can be successfully resisted by the material of which the canal perimeter is constructed, and what other factors may be present in the future to cause this critical tractive force to be higher or lower?" The other factors may include vegetative growth on the canal bed and banks, canal structures such as drops, chutes, bridges, and checks, and the number and degree of bends. These factors would tend to increase or decrease the forces of resistance to the forces which cause scour.

Designing a channel that will be stable in an alluvial-type material, or stabilizing a channel that is already in a state of deterioration, poses many problems. There are several methods of approach that can be utilized, but the selection and application of any method is dependent upon the engineers' judgment and the existing conditions. The procedures advocated by various engineers can serve only as guides and none of them are completely applicable to every situation. P. W. Terrell and W. M. Borland[25] have presented some of the requirements and Bureau experience in the design of earth canals and other channels. Some of the more familiar publications on stable channel design are those by Lane,[18] Lacey,[26] Blench,[27] and Maddock and Leopold.[28]

The Lacey and Blench equations for stable channel design are developed from conditions in canals and channels in India where conditions of flow, bed and bank materials, and channel hydraulics differ from those encountered in the western United States. Their equations also require considerable judgment or basic assumptions on the part of the designing engineer. This may be difficult where insufficient data or knowledge of the particular channel are available to influence adequately the engineers' judgment or assumptions within a reasonable degree of accuracy. Lane's design criteria consider the tractive forces acting on the periphery of a canal. The Maddock-Leopold

relationships require knowledge of certain factors such as sediment transport and settling velocity of the sediments in transport that may not be available or are difficult to determine. These relationships are based on analyses of many streams in the western United States and are overall concepts that may not always be applicable to a particular reach of a given stream. The Maddock-Leopold equation currently in use by the Bureau of Reclamation is as follows:

$$\frac{W}{d} = \left[\frac{\frac{S^{1/2}}{n}}{0.225(C_s V_s)^{0.395}}\right]^3 Q^{0.555}$$

where

Q = discharge in cfs
V_s = weighted mean settling velocity for total sediment load
S = stream gradient
C_s = total load sediment concentration
n = Manning roughness coefficient
W = channel width
d = channel mean depth

In this equation the sediment concentration is required and is therefore unsatisfactory for clear water conditions such as may occur below a dam or where the sediment load is eliminated by channel rectification.

Research is currently being carried out by the Bureau of Reclamation using data obtained in the field from existing canals, drains, and other natural and artificial channels, to evaluate more clearly the requirements for stability. Of particular concern are the requirements for stability in materials within the coarse silt and fine sand bracket where there is a transition from cohesive to non-cohesive conditions.

In the design of a stable channel, one of the first factors that must be determined is the so-called "dominant discharge." This might be defined as that discharge of a natural channel which determines the characteristics and principal dimensions of the channel or is most influential in dictating the general condition of stability. In some cases it is easily determined, whereas in others it is not. Many rules have been presented to arrive at this discharge, none of which necessarily give the same answer. We find that the dominant discharge varies with the sediment characteristics, with the relationship between maximum and mean discharge, with the shape of the flow duration curve, and with the flood frequency. The establishment of the dominant discharge is still one of the most difficult problems in channel design work.

Another factor that is not always easily evaluated is the size distribution of the bed materials under design conditions. Will it be the same as for the channel before rectification, or will it have an entirely different size distribution curve?

Still another factor that enters into the design is the probable sediment transport under stabilized conditions. For example, when the sediment in transport is being derived from the bed and banks what will be the transport conditions and the channel requirements for such transport after rectification reduces or eliminates the source of sediment as compared with the design

based on existing conditions of sediment transport?

The above discussion points out only some of the elusive factors in channel design work. Engineers working on channel design problems do the best they can with the tools available and attain a surprising degree of success considering the inadequacy of these tools, but the need for more exacting methods of design is evident.

Deposition Behind Structures

Distribution of sediment in reservoirs. One of the more common problems with which the sedimentologist is faced is the prediction of how the sediment will be deposited within a proposed reservoir. This prediction is necessary to aid the designers in the location and type of outlet works and to determine the probable loss of storage capacity within the various storage allocations over a given period of time. Most of the present procedures used to predict sediment distribution are based on information obtained from resurvey of other reservoirs. The old assumption that all the sediment will deposit in the bottom of the reservoir has been refuted.

The sediment distribution in reservoirs is primarily dependent upon the following:

a) Type of reservoir operation
b) Reservoir configuration
c) Size distribution of the inflowing sediments
d) Flood detention period

There are, of course, other factors that influence the distribution to varying degrees such as the trap efficiently, narrow necks within the reservoir area, vegetative growth in the delta area, heavy sediment contributing tributaries within the reservoir area, and density currents.

The publications available on sediment distribution do not completely evaluate the above-listed major items that influence distribution. Herein lies the need for research that can result in better procedures for predicting how the sediment will be deposited. A publication by the Bureau of Reclamation which is in the form of a progress report(29) has attempted to evaluate some of the influencing factors. Evaluation of other factors is still dependent to a large degree upon the engineers' judgment. Work is being continued to improve the procedures presented in these reports. Of course, the prediction of sediment distribution is not an important item in all reservoirs. In those where it is important, but not a major concern, rapid methods of approximating the distribution can be utilized.

We believe that there is sufficient data available on numerous existing reservoirs to correlate the influencing factors and arrive at more accurate methods of predicting sediment distribution. Assembling and evaluating these data would be a noteworthy contribution to the field of sediment hydraulics.

Deposition above spillway level. In computing a backwater curve above an obstruction in a river channel, one must assume a discharge, river cross sections, and value of channel resistance which can be used in conjunction with some stream flow formula. If the stream is a heavy sediment carrier the backwater curve will change with time as sediment deposition takes place. The rate of sediment deposition is dependent on the reduction in velocity. Locally this can be caused by a wide variety of obstructions that occur or are built into the stream channel, see Figure 10.

Density currents. Much literature has been devoted to the subject of density currents as indicated in the Bibliography of the Bureau of Reclamation Hydraulic Laboratory Report No. Hyd-373.(30) Experience has shown, in the United States, that very little sediment has passed through reservoirs as density currents, particularly as the reservoir becomes older. The reason for this seems to be the fact that the surface width of the sediment deposit becomes wider and wider as more sediment deposits in the reservoir. The turbidity current decreases its velocity and less sediment reaches the dam. Also, in cases where density currents do occur it may not be possible to release water to take advantage of the density currents.

Effect on intakes at diversions. Sediment deposition behind diversion structures on alluvial streams occurs rapidly as soon as the diversion dam is built. In at least two cases on the Republican River—Superior-Courtland and Cambridge Diversion Dams—the first floods occurring after the dams were completed practically filled their pools with sediment. It is common practice in the Bureau of Reclamation to include a gated sluiceway providing a low-level opening through the dam near the canal intake. One of the vital purposes of this sluiceway is to maintain an open channel leading to the canal headworks during the recession of a flood. Instructions are issued to operators in the field suggesting the sluice gates be opened during a receding flood in order to insure an open channel to the headworks. For low diversion dams, sediment deposited during a flood could very easily block off a canal headworks, causing the normal river flow to go over the spillway which may be a few feet higher than the headworks crest. After diversions are constructed the best operation criteria are developed by testing and utilizing the operation criteria supplied by the designers.

Where the dam is of low-head type and the sluiceway crest is only a few feet below the headworks crest the sediment concentration in the sluiceway and headworks is practically the same for continuous sluicing. Model studies on the Bartley Diversion Dam(31) showed intermittent sluicing increased the total sediment load through the sluiceway many times over that for continuous sluicing.

Channelization through depositional areas. One of the most perplexing problems that is associated with sediment deposition behind structures, particularly in the western United States, is delta development. The channel in the backwater area of a reservoir becomes plugged with a combination of sediment deposits and vegetation which is generally in the form of water-loving plants such as phreatophytes and tules. This in turn causes troubles upstream, especially during flood stages, such as interference with upstream structures and diversions, clogging of drain and sewer outlets, water loss to the water-loving plants by transpiration, and increased evaporation losses. The problem has become so acute in some delta areas that it has become necessary to take steps to relieve the situation. Typical examples are the Pecos River above MacMillan Reservoir and at various locations on the Colorado River. So far no completely effective method has been devised to annihilate the water-loving vegetation and there is no practical way to prevent the development of deltas.

The problem then is to convey the water and the sediment through the delta area without sediment deposition and with a minimum loss of precious water. The procedure generally followed to accomplish this is to construct a designed channel and then maintain that channel so that it will accomplish the

job for which it was created. The methods used in the design of a channel such as this have been discussed elsewhere in this paper. In the Middle Rio Grande Valley the Bureau of Reclamation has utilized a separate conveyance channel and floodway and on the Colorado River a combined floodway and conveyance channel.(32)(33) These channelization works have not been observed under flood flow conditions as yet but the ability of a river channelization to withstand flood flow forces is the ultimate test of their adequacy. The need for adequate tools with which to approach these delta problems is evident. How can we effectively eliminate the phreatophyte and tule growth which is taking such a large toll in water in an area of the country where all the water is needed for beneficial use? How can we create a channel through the delta that will prevent the adverse developments upstream and at the same time be economically and engineeringly sound? Much progress has been made in finding the solution to these problems in recent years but considerable research and development remain to be undertaken.

Channel Degradation

Release of clear water from a dam or introduction of irrigation return flows to a channel usually result in a degradation cycle if the channel consists of unconsolidated materials. Degradation below Davis Dam is shown on Figure 10. It is necessary for the engineer to predict when and to what degree this degradation, with accompanying water surface lowering, will occur and how it affects the tail water curve for a dam site and installations downstream.(34) No clear cut procedures are available at the present time on which to base such a prediction although many cases have occurred in which there has been degradation below hydraulic structures. The amount of degradation is dependent upon several factors which include the variation in size of bed material with depth, channel hydraulics, flow duration pattern, and the existence of downstream controls. Degradation may be limited in some cases only by the amount of storage behind a structure or the limiting slope for transport of the materials in the channel. In other cases it may be limited by the creation of bed armor. All present methods of computing degradation are affected by the empirical procedures for determining transport and limiting size of transportable materials.

Of particular concern in the western United States is the effect of irrigation wastes and return flows on a natural channel which is being used as a project wasteway or drain. This usually creates a different flow duration pattern than the one on which the channel was developed and a greatly increased "dominant discharge." Many such channels are changed from ephemeral to perennial(23) by injecting return flow from project wasteways or drains.

Research that will result in more reliable methods of sampling bed material, computing bed material transport, and determining limiting tractive forces will reflect directly into the accuracy in predicting degradation. More research is also needed to develop procedures for computing the channel degradation. Perhaps a correlation of records of actual degradation with empirical computations for determining bed material transport could be accomplished to devise procedures for predicting degradation below structures.

Sediment Control

In the development of Bureau of Reclamation projects, sediment studies based on sediment and water measurements are made to give the best estimate of sediment quantities involved. Then with the aid of model studies, the designer includes a sediment control device in his design. For the coarser bed sediments these sediment control devices have included curved guide walls, vortex tubes, and short tunnels.[35] For finer sediments a settling basin type of control is usually used in conjunction with mechanical or hydraulic sediment removal equipment. Two factors that influence the decision as to the type and extent of sediment control facilities included in the design of a diversion are: (1) the size and amount of sediment needing control and (2) the cost of sediment control versus benefits derived.

A typical study is that of Milburn Diversion Dam on the Middle Loup River in Nebraska.[36] A sediment study showed the diverted sediment load could be as much as 600 acre feet per year without sediment control. A hydraulic model study developed a short tunnel type of sediment control device, based upon which the designers recommended a diversion including a short tunnel and a settling basin for desired sediment removal. The short tunnel is very effective because it divides the water on a horizontal plane, allowing the water with only light suspended load to enter the canal, while the bedload and lower part of the suspended load pass through the short tunnel to the river channel downstream.

In those cases where excess sediment is carried into canals, various devices have been used to remove the coarse bed sediments. In the countries of India and Pakistan these are called excluders. Ralph L. Parshall[37] developed the vortex tube for removing sand from canals. His studies and later studies by Koonsman[38] indicated that to get good action in the vortex tube, a velocity near critical velocity $V = \sqrt{gd}$ was required. To overcome this difficulty of requiring such a high average stream velocity to get good action, an activator to work in conjunction with the vortex tube was developed in the Hydraulic Laboratory of the Bureau of Reclamation as a part of the model studies for Republic Diversion Dam. This activating vane is placed directly over the vortex tube with tapered upstream and downstream portions so that the velocity is increased as it approaches the vortex tube. The necessary drag force is thereby created on the channel bottom to create good vortex action in the tube keeping the bed material in suspension so it can be drawn off. Where the drag forces are insufficient, the vortex tube will fill with bed material and the tube will become ineffective as an excluding device. A typical design for a vortex tube with activator is shown on Figure 11. Many other types of sediment control devices on canals have been tried in this country and particularly in India and Pakistan. The success to which these work varies a great deal and depends upon the hydraulic characteristics of each case. The philosophy in this country has been to remove most of the unwanted sediment load with the least amount of water loss. Research along this line of thinking is needed to improve the design and operation of our diversions and canal systems.

SUMMARY

In the foregoing discussion some of the sediment problems encountered by Bureau of Reclamation engineers in the planning, design, and maintenance of

irrigation projects have been pointed out and some of the methods of solution have been indicated. The need for research in sediment hydraulics to improve and simplify present methods of acquiring basic data, computation, and design is apparent. There is a wealth of existing data available in the United States and abroad that could be combined with additional field and laboratory research to advance the science of sedimentology.

In some sections of the world, where irrigation developments are centuries old, sediment hydraulics has been and still is one of the major engineering problems. This points up the need for recognizing these problems early and developing corrective measures to reduce sediment problems before they become enormous.

The following is a summary of the phases of sediment hydraulics discussed herein where research is needed:

Sediment sampling—Improvement and simplification of present sampling equipment, procedures, and techniques will reduce the cost of a sediment sampling program and at the same time produce more representative data. Work is continually being done by such organizations as the Federal Inter-Agency River Basin Committee to improve sampling equipment. This work should be continued.

Bed material transport computations—The need for accurate and versatile methods of deriving unmeasured sediment transport is ever present. New formulas can be developed and present formulas improved so that they can be used with confidence within defined limitations. In developing any sediment transport formula, the point must be kept in mind that its usefulness depends upon the accuracy of the answers given when computing transport of natural streams.

Transport in nonalluvial streams—Existing bedload formulas were developed for alluvial-type channels with bed material of sand and fine gravel sizes. There are no such formulas for determining the transport in mountainous-type streams that are not strictly alluvial and have bed material ranging from sands to large boulders. Can a method of computation be developed to fit this case or should efforts be directed toward actual bedload sampling? If a bedload formula can be developed, how should a representative sample of the bed material existing under various river stages be obtained?

Total sediment transport—Because of the mechanical limitations of our present day sampling equipment and the early day developments of formulas to compute transport, we tend to think of sediment transport as segments rather than the overall total load. Total load and its size distribution is what must be determined in sediment studies. Inclusion of total sediment load figures in the published records would be a notable improvement and would be of much greater value to the sedimentologist.

Stable canals and channels—The need to define the influencing variables that create a stable condition in a canal or channel is one of the most urgent needs in the field of sediment hydraulics. Various formulas have been developed for computing stability but application of these formulas to a variety of field problems has indicated that further research is needed. Research along this line is being carried out by some organizations and agencies both in the field and in the laboratory.

Design for stability—The definition of the correct combination of variables that result in canal or channel stability will culminate in reliable methods for designing for stability. Although current design formulas and procedures appear to conflict or are based on a somewhat different approach it is hoped that continued investigations and research will reduce these differences and result in procedures that can be combined with definable field data to give more accurate solutions. The criteria for the type of channel control to use under different conditions and the limitations of these control works should also be improved and be more firmly established.

Aggradation and degradation—Very little progress has been made in developing methods for predicting channel aggradation and degradation. It is believed that a combination or correlation of records on actual happenings with sediment transport theories could provide some procedures for predicting aggradation and degradation. These procedures will, of course, still be limited in accuracy by the adequacy of the bedload formulas.

Deposition in reservoirs—Methods of predicting the distribution of the sediment that is to be deposited in a reservoir are still in need of improvement. A vast amount of data are available on existing reservoirs that could be sorted out and combined, perhaps by multiple correlation, to derive procedures or formulas for predicting how the sediment will deposit behind a structure.

Diversion structure design—The design of a diversion structure on a stream carrying unwanted sediments usually includes a sediment control device. The method employed to exclude the sediments can best be determined by a laboratory model study. Model study results, combined with the results of the prototype operation, will yield even better designs in the future.

Stabilizing canals below structures—The use of riprap below structures in canals is a common design practice to prevent or repair cases of erosion. The size and amount of riprap used are usually chosen on the basis of experience. Research which would assist in this field is needed.

Sediment exclusion from canals—Reducing the cost of cleaning canals is a goal towards which operating engineers on all irrigation projects are continually striving. Hydraulic sediment excluding devices are in use in some locations but for the most part canals are usually cleaned with mechanical equipment. Development of less costly means of keeping undesirable sediment out of canals would be very welcome research.

Design of canals in cohesive soils—The design of stable canals in earth materials to prevent scour necessitates the balancing of the scouring forces with the resistive forces. For noncohesive materials considerable information is available. There is a definite need to determine a correlation between some soils tests for cohesive materials with their resistance to scour by flowing water which can be used as a guide for designing canals in cohesive materials.

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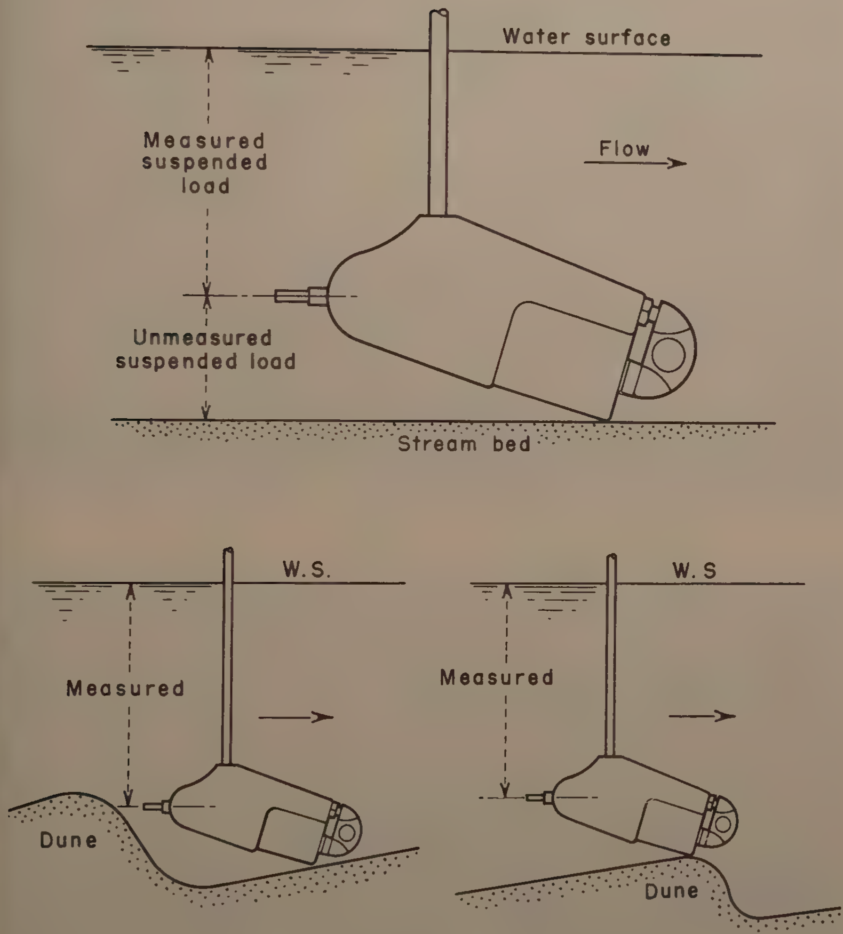
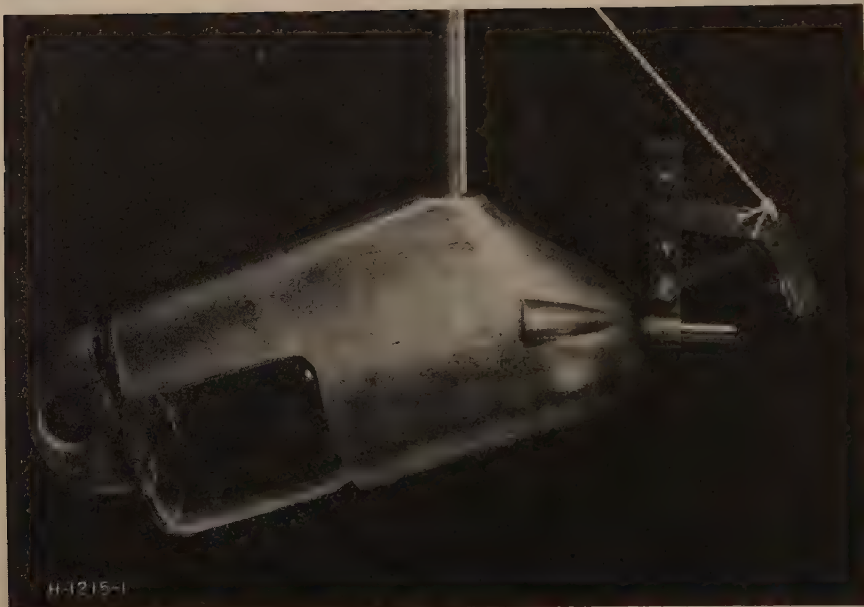


FIGURE 1 – SAMPLING LIMITS - D.H.- 48 DEPTH INTEGRATING HAND SAMPLER



A. DH-48 Sampler with nozzle adaptor for point sampling.

B. Plastic Tube Sampler for sampling close to hard stream bottoms and for large sediment.

Figure 2. Samplers adapted for special conditions.

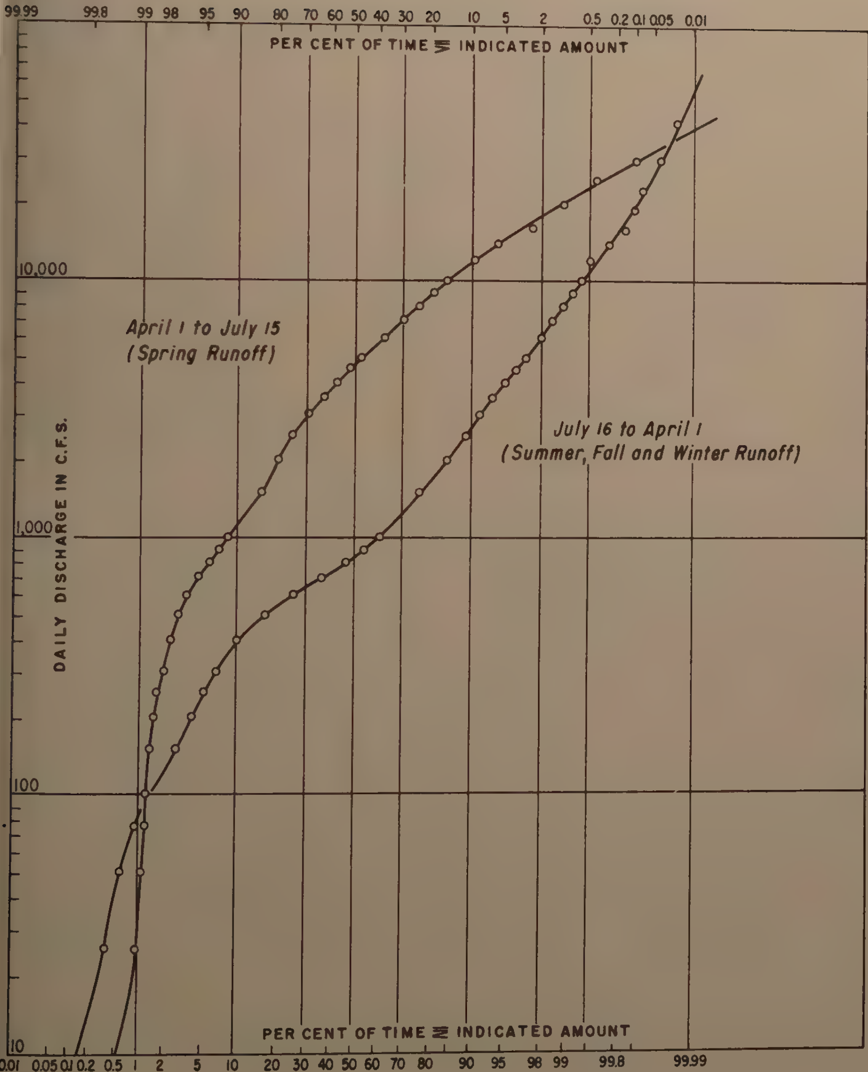


Figure 3 – Seasonal Flow-Duration Curves,
San Juan River, Bluff, Utah

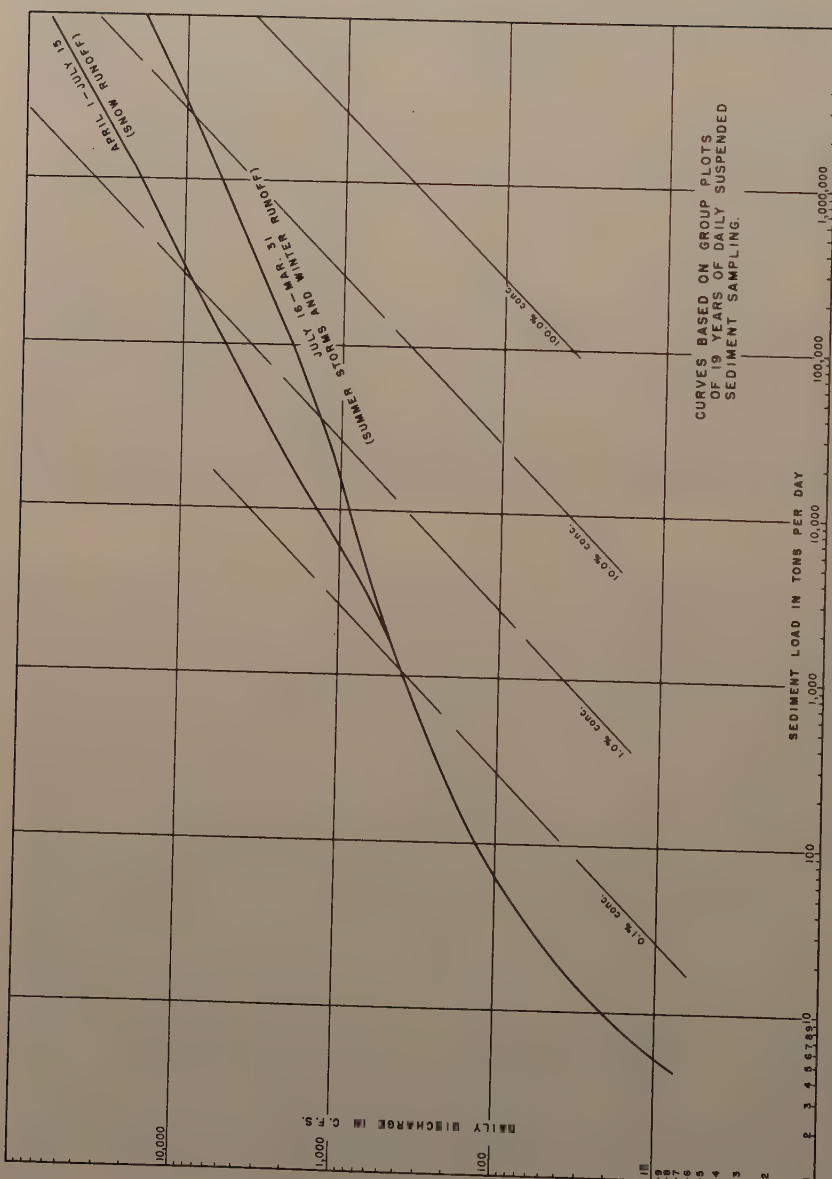


Figure 4 —Seasonal Suspended Sediment-Rating Curves,
San Juan River, Bluff, Utah

Station: Bluff, Utah Period: April 1 to July 15 River: San Juan

1 % limits	2 % interval	3 % mid ord	4 Q_w	5 Q_s	6 2 x 4 Q_w disch	7 2 x 5 Q_s disch
0.00-0.02	0.02	0.01	40,000	3,500,000	8.0	700
0.02-0.1	0.08	0.06	32,000	2,400,000	25.6	1,920
0.1 -0.5	0.4	0.3	25,000	1,550,000	101.2	6,200
0.5 -1.5	1.0	1.0	20,700	1,130,000	207.0	11,300
1.5 -5.0	3.5	3.25	16,300	740,000	570.5	20,450
5-15	10	10	11,700	400,000	1,170.0	40,000
15-25	10	20	8,900	245,000	890.0	24,500
25-35	10	30	7,000	157,000	700.0	15,700
35-45	10	40	5,650	107,000	565.0	10,700
45-55	10	50	4,720	79,000	472.0	7,900
55-65	10	60	3,750	52,000	375.0	5,200
65-75	10	70	2,940	33,500	294.0	3,350
75-85	10	80	1,950	17,000	195.0	1,700
85-95	10	90	1,100	7,000	110.0	700
95-98.5	3.5	96.75	540	2,100	18.9	74
98.5-99.5	1.0	99.0	25	12	0.3	
99.5-99.9	0.4	99.7	8	5		
99.9-99.98	0.08	99.94	2	2		
99.98-100	0.02	99.99	0	0		
				TOTAL	5,702.5	150,394

Q_wP.D. = 5,702.5 D.D. x 106 x 1.9835 = 1,200,000 (AF)/period
Q_sP.D. = 150,394 D.D. x 106 = 15,950,000 tons/period
20 percent correction for bedload = 3,200,000 tons/period
Total sediment discharge = 19,150,000 tons/period

D.D. = daily discharge
P.D. = period discharge
D.A. = drainage area 23,000
62.5 = lb/cubic foot
1,361 = tons/acre foot

Sediment

P.D. $\frac{19,150,000}{1,361}$ $\frac{\text{tons/period}}{\text{tons/(AF)}}$ = 14,070 (AF)/period

Yield $\frac{19,150,000}{1,361 \times 23,000}$ $\frac{\text{tons/period}}{\text{tons/(AF)} \times \text{D.A.}}$ = 0.612 (AF)/sq mi

Concentration $\frac{15,950,000 \times 100}{1,200,000 \times 1,361}$ $\frac{Q_s\text{P.D.} \times 100}{Q_w\text{P.D.} \times 1,361}$ = 0.977 percent

Runoff

Rate $\frac{1,200,000}{23,000}$ $\frac{Q_w\text{P.D.}}{\text{D.A.}}$ = 52.17 (AF)/sq mi

Figure 5.--Sediment Load for San Juan River at Bluff, Utah, Spring Runoff Period

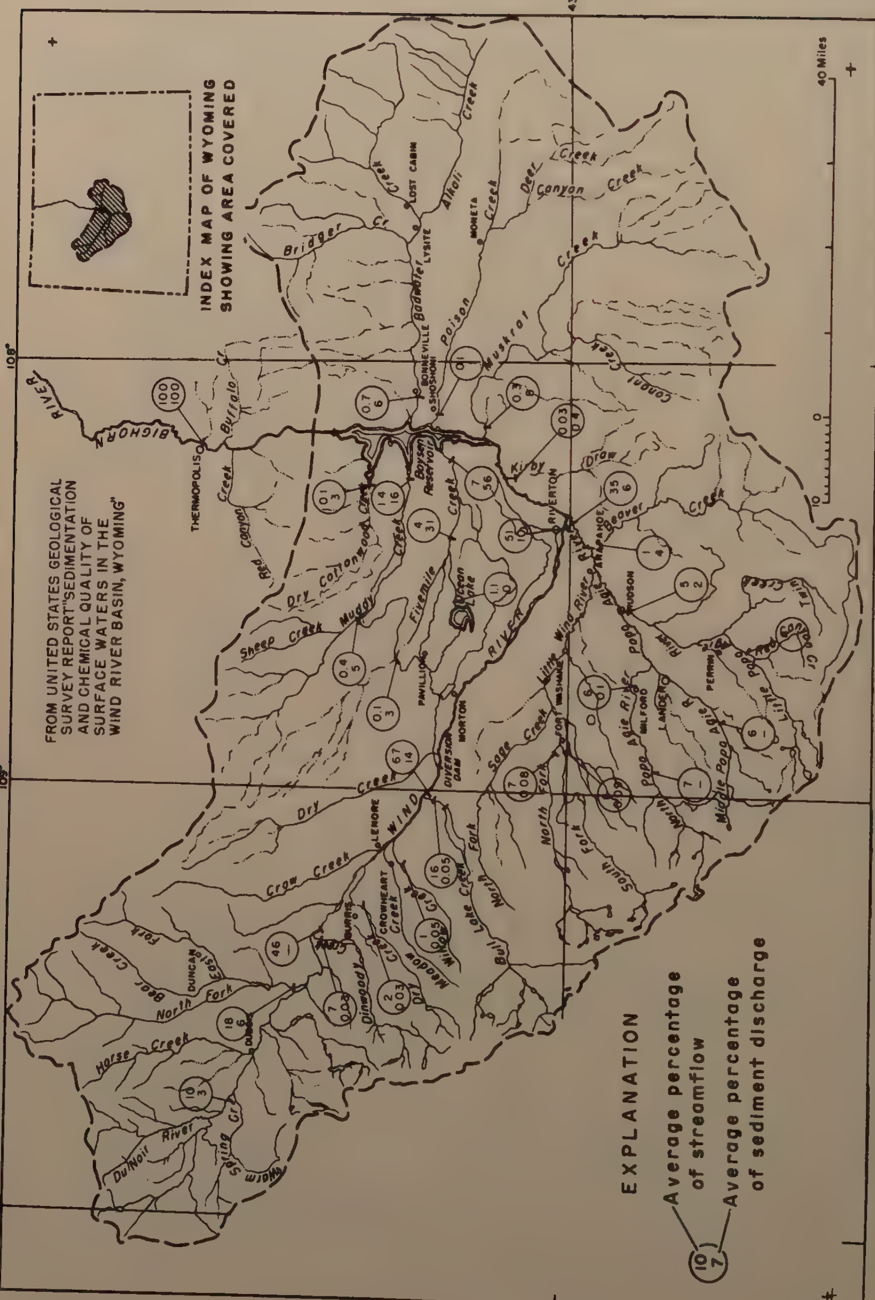


FIGURE 6 APPROXIMATE AVERAGE SEDIMENT DISCHARGE AND STREAMFLOW IN PERCENTAGE OF SEDIMENT DISCHARGE AND

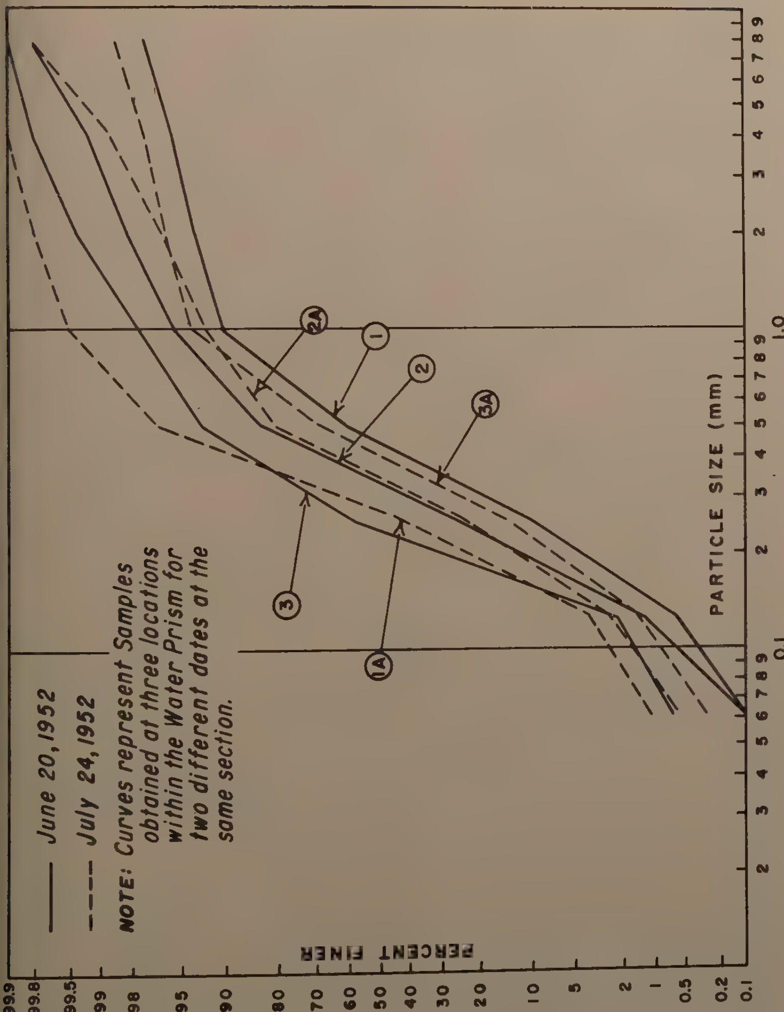


Figure 7 - Bed Material Size Analysis Curves, Middle Rio Grande River, Bernalillo Study Reach, Section F

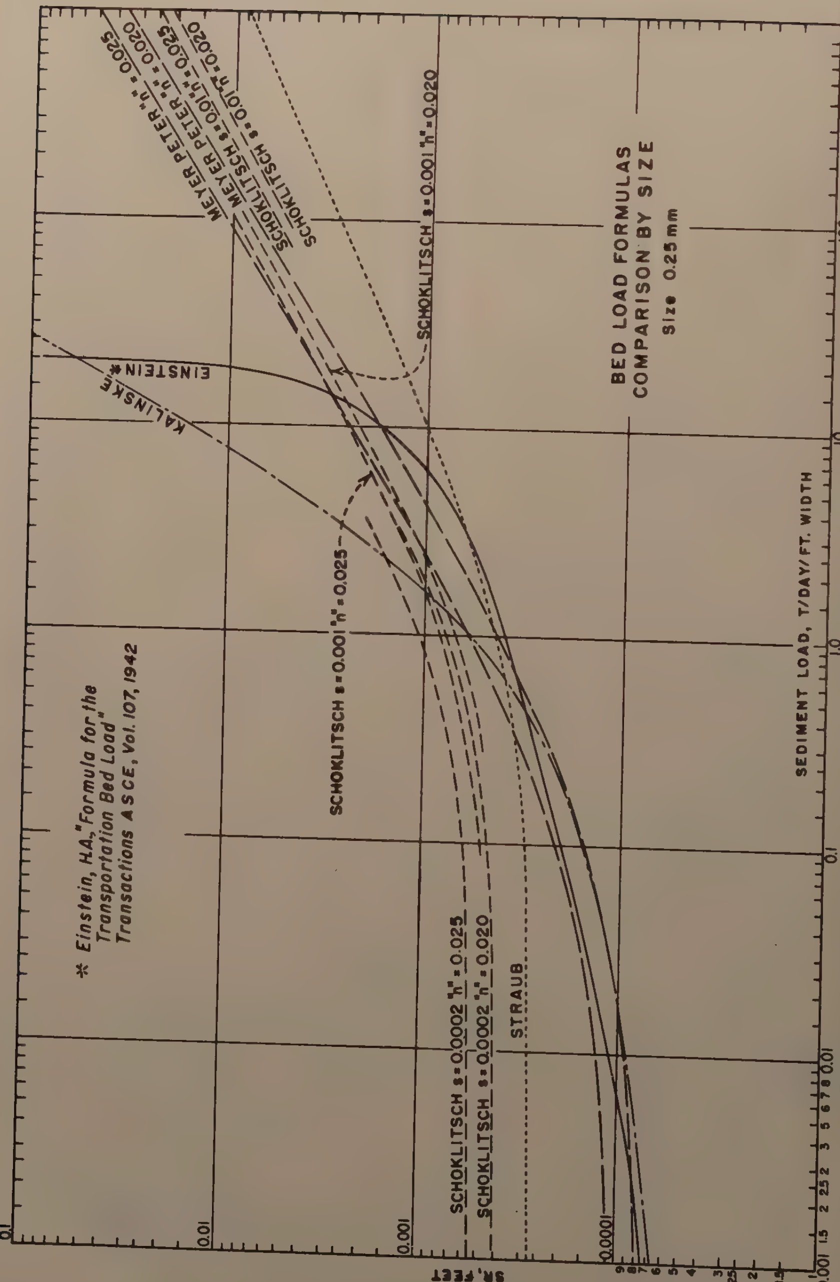


Figure 8 - Showing Variation of Transport as computed from various Formulas

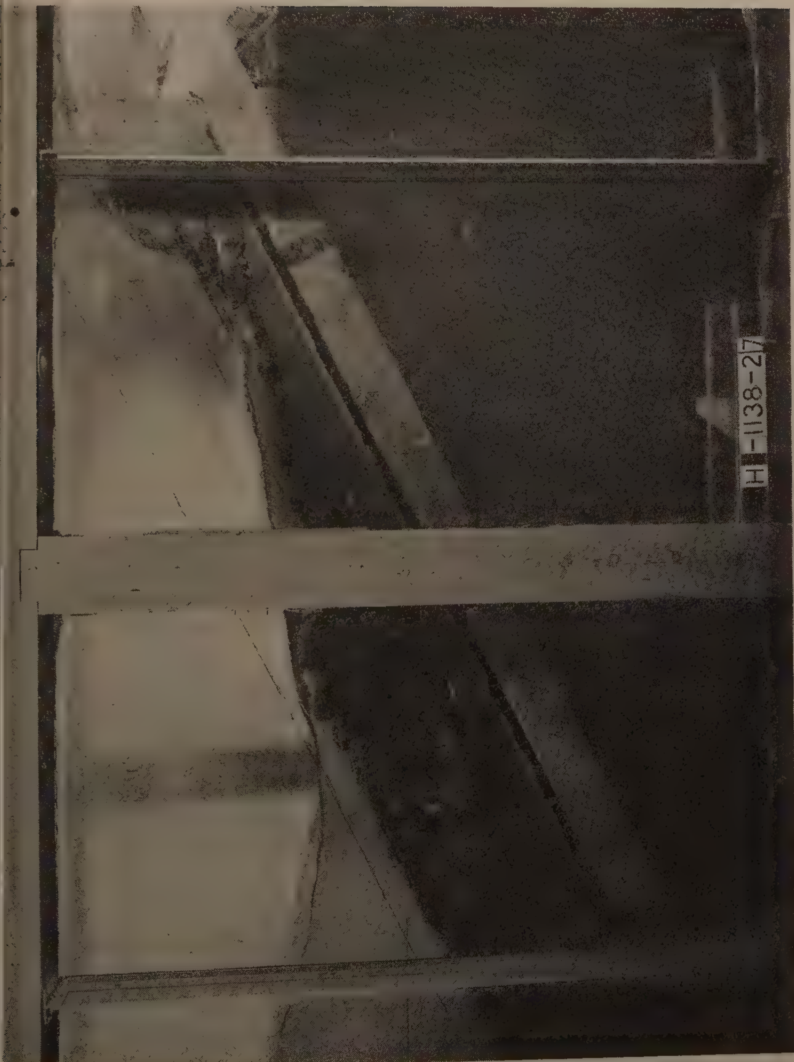


Figure 9. Beaching slope of 5.14 to 1 developed on pit run gravel from Kennewick Main Canal, Yakima Project Washington, after being subjected to 0-43 foot high waves at a frequency of 42 per minute for 19 hours.

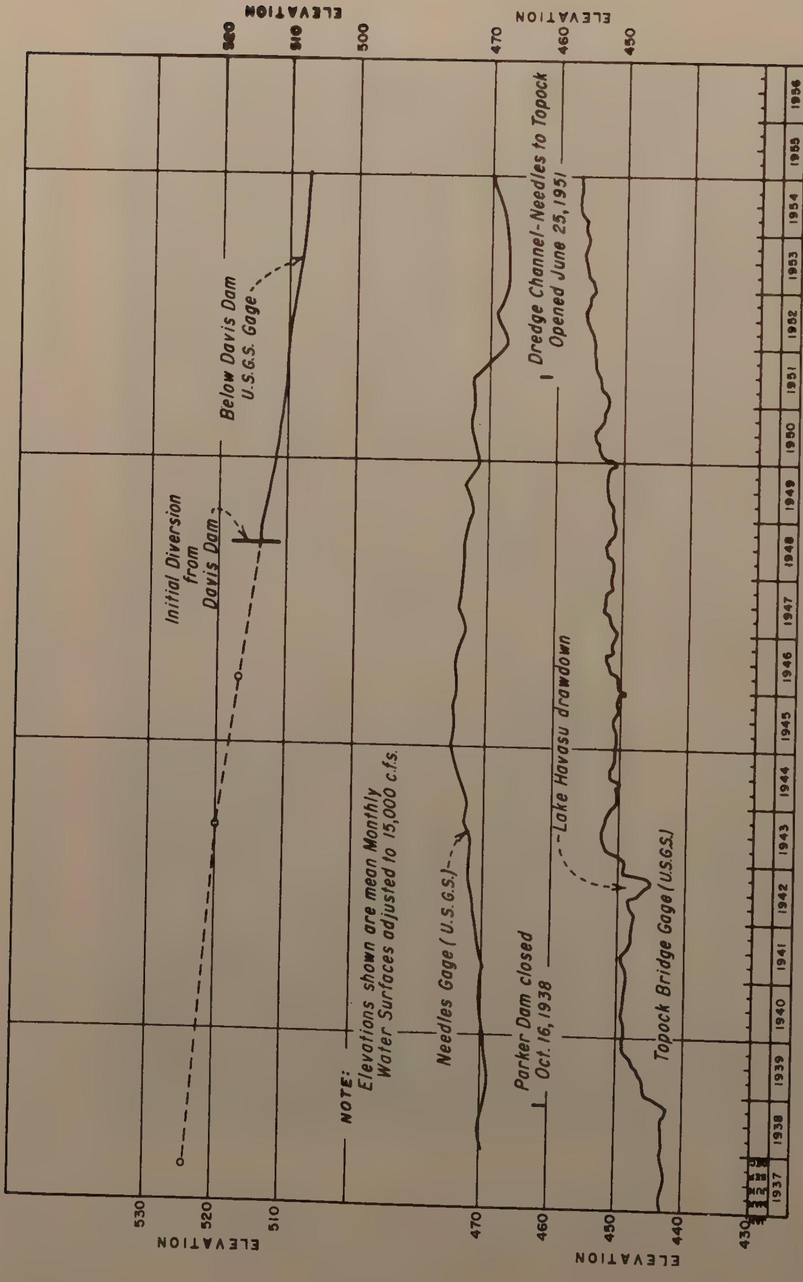
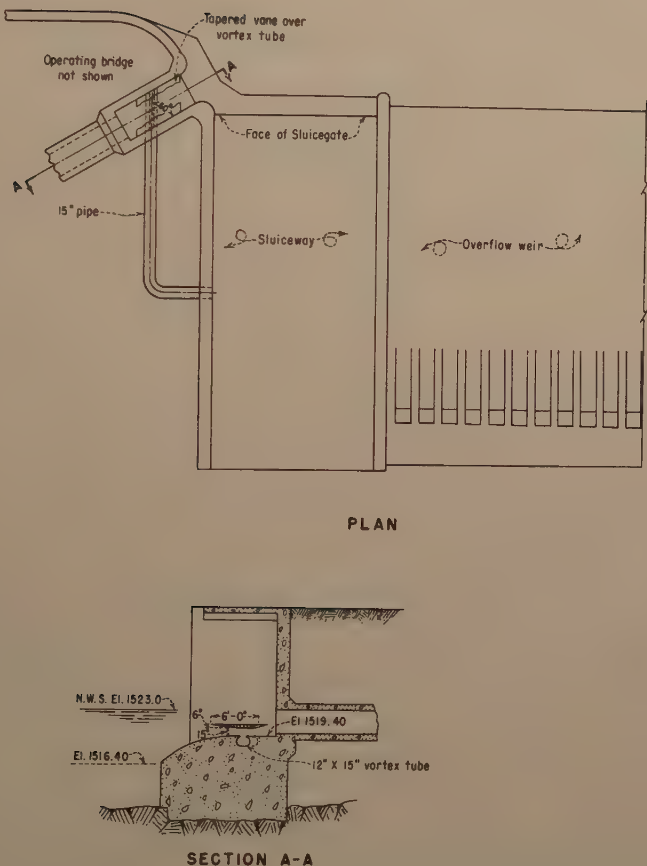


Figure 10—Change in Water Surface Elevations due to Aggradation and Degradation between Davis Dam and Lake Havasu



HYDRAULIC MODEL STUDIES
REPUBLIC DIVERSION DAM
RECOMMENDED DESIGN-HARDY HEADWORKS
MODEL SCALE 1:15

Figure 11 – Vortex Tube with Activating Vane for Sediment Exclusion

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FLOOD PLAIN ZONING AS SUPPLEMENT TO FLOOD CONTROL[1]

Emil P. Schuleen,[2] M. ASCE
(Proc. Paper 954)

SYNOPSIS

Economic considerations may limit the degree of flood protection that may be provided and thus leave some areas subject to residual flood damages. Unless this limitation is fully understood by all concerned, progressive expansion in the flood zone may eventually reach such proportions that additional flood control measures will be required to protect the added wealth placed in the path of the flood waters. Flood plain zoning offers a means of controlling development within the flood zone to the degree that can be tolerated without increasing the frequency of excessive flood damages. Zoning may be accomplished by local ordinance, by purchase of fee title to some areas and flowage easements over others, or by general education of the public regarding flood dangers that still exist in the area.

INTRODUCTION

Flood plain zoning provides a basis for coordinating man's use of the flood zone with that of the river. It seeks to limit the intensity of use to those activities and facilities that by their very nature must be located in that region, and to limit other developments to the types and extent that can tolerate the flood potentiality that may be expected at their respective locations. The procedure may be used as a substitute for flood control measures, or as a supplement to such facilities. In this paper, attention will be directed primarily toward the latter use. Attention furthermore will be directed toward consideration of urban areas, rather than rural or agricultural areas, although it is significant to note that the same principles would be applicable in either case.

In a broad sense, occupation and use of the flood zone represent an encroachment on the river's domain. This encroachment may be considered as

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of two types. The first is that ordinarily envisioned when the term flood plain is used in connection with the placement of structures and other facilities in the region between the top of the river bank and the landward limit of the area subject to flooding. Since occupation of this type is outside of the normal flow channel of the stream, it generally has little effect on the flood itself, except insofar as the volume of overbank storage normally available for flood discharge is eliminated by structures. The principal effect of this type of occupation is that it places material wealth in a position where flooding of the stream will cause loss or damage to property.

The second type of encroachment is that where the stream bank or bed is occupied by structures. Examples would be where the river bank is extended by filling in order to gain area at or above the top-of-bank level, or where structures, such as bridge abutments, and even buildings are so placed as to encroach seriously on the waterway capacity of the stream. It should be noted that when structures, particularly buildings, are placed over or encroaching on the flow channel of the stream, not only is wealth placed in the pathway of the flood water, but the structures themselves may actually impede the movement of flood water and thus aggravate the flood condition upstream.

History of Development

As a background to consideration of the problem of flood plain zoning, it may be well to review briefly the history of development within the flood zone of our streams. The need for use of the stream for various functions has, since the earliest days of settlement, resulted in a concentration of development along the stream banks in urban areas. These essential uses of the stream are water supply, both domestic and industrial; waste disposal; transportation, both on the waterway itself, and by railroads and highways, the latter due to the more favorable grades available on the flood plains, particularly in the more rugged areas of the Country, such as western Pennsylvania and West Virginia. A fourth use, which was of considerable importance when the Country was young, was that of water power for saw and grist mills, the location of which served to concentrate community development in the area of the mill.

The primary cause of encroachment on the flood channel itself, has been the progressive need for expansion of properties and the general congestion of the urban areas, which would permit expansion only in the direction of the stream.

Several other factors have influenced growth of development in the flood zones. The pioneer, of course, lacked information regarding the flood potentialities of the stream, other than could be discerned from physical evidences of past floods. Areas that were subject to more or less frequent flooding naturally were developed with full knowledge of the flood danger. The intervals between floods of greater magnitude, however, have been the cause of much development in the flood zone that might not otherwise have taken place. For example, there has always been a tendency to forget the flood as the time since the last experience lengthens. Furthermore, where properties have changed ownership since the last severe flood, the new owner quite often has acquired the property and developed it without full knowledge of the flood danger. In general, therefore, past history has indicated that following a severe flood, there is a period of depressed values and caution in use and

development of the flood zone. However, as time passes without severe flood, the tendency toward caution lessens and property values gradually increase.

Through the action of these various factors and the general growth and expansion of the Nation, there now are immense values in physical development within the areas along the stream banks that in the past have been subjected periodically to floods of various magnitudes. With the flood of 1927 in the lower Mississippi River, the Nation was alarmed sufficiently that Congress, by the Flood Control Act of May 1928, authorized a project of expanded scope to provide adequate flood protection for the lower Mississippi River. Approximately nine years later, in March 1936, there occurred floods in the northeastern section of the United States, including the upper Ohio Valley, of such magnitude as to exceed any known flood that had previously occurred in the respective streams. This, in turn, spurred Congress to pass the Flood Control Act of June 1936, in which it recognized the flood control problem in general as one of national interest. Subsequent to these two flood control Acts, there have been various others and various flood control measures have been passed by States and local governments, with the result that the provision of flood control facilities in areas of need has been well advanced, a number of such projects having already demonstrated their value.

Limitations of Flood Control Facilities

Many of these flood control projects, however, are limited in effectiveness, a condition of which the general public is not fully aware. Where levees and flood walls have been constructed to protect urban areas along the major streams, they generally have been designed to provide a high degree of protection, in many instances protection against the maximum possible flood or at least against a flood of such proportion as would approach that limit. Where measures have been constructed to provide protection of that degree, it may be stated that for all practical purposes, the flood zone within the protected area has been eliminated. In certain instances, however, particularly in the headwater areas, it is not economically possible to provide levees or flood walls of such a height as would protect against a flood approaching the maximum possible. In these cases, the flood zone remains, but it is not subject to inundation, except during very extreme floods.

Where channel improvements are provided, they are effective, at least to some degree, on floods of all magnitudes, but it is seldom economically possible to provide protection of this nature to the extent that the maximum possible flood will be contained within banks. Such a limited channel improvement, however, will eliminate floods along the extreme outer edge of the flood zone and make less frequent the floods in the remainder of the flood zone. It is evident, therefore, that there remains in this case also, a residual flood potentiality.

Where storage reservoirs are provided as a means of flood control, it is seldom economical to provide storage capacity sufficient to control completely the runoff from the maximum storm that could occur over the tributary area. However, the reservoir will provide complete protection immediately below the location of the dam for all except those flood runoffs which exceed the capacity of the reservoir. The effectiveness of a reservoir becomes progressively less with distance downstream from the location of the dam. This is due primarily to the lessening in the percentage of area controlled and to deviation in the synchronization of the maximum effect from the reservoir

storage with the flood runoff from the drainage area downstream from the dam. A reservoir provides some reduction in flood height for a considerable distance downstream, but due to the economic limitation of the structure, there remains as in the case of the limited channel improvement, a residual flood potentiality. This potentiality may, of course, be further reduced by the addition of other reservoirs at strategic locations and by supplementation with flood walls, levees, and channel improvements where such can be found to be economically justified.

It should be evident, however, that even when all of the flood control measures now planned are completed, there will still be areas where complete flood control or flood protection has not been provided. It is in connection with these areas in particular, that this paper is concerned. If the previously mentioned factors contributing toward the occupation of the flood zone are reconsidered, it will be evident that many of them will still be effective. While domestic water supply in a modern community may require only limited facilities along the immediate river bank, the need for industries to be near the stream as a source of water supply will continue to be of consequence and even more so than in the past. With the growth of modern industry, the demands for water supply have multiplied many fold during recent years. These needs may be expected to continue and even increase. On the other hand, modern waste disposal facilities to the extent that they must occupy the flood zone, may be designed in such a manner as to minimize damage from flood. In the field of transportation, however, particularly on the navigable streams, traffic has increased tremendously over recent years and may be expected to continue. Factors contributing toward the occupation and use of the flood channel will also continue to be effective if not controlled.

In addition to these factors, the general public and even some public officials are not fully aware of the limitations in effectiveness of such flood control measures as have been provided. There has grown up a feeling that with flood control as now provided, areas that were formerly subject to flooding are now flood-free. Furthermore, civic organizations in their zeal to attract new industries and commercial enterprises to their respective areas, have often over-stressed the safety feature against floods.

Need for Supplemental Flood Plain Zoning

With these various factors still operating, there is danger that unless controlled, continued development in the flood zone will eventually result in the need for even further flood control measures to eliminate damages created by the additional encroachment within the flood zone. It may be well to stress at this point, that it is not intended to imply that further development within the flood zone should be completely prohibited. Certainly, the provision of flood control measures has made more useful, land which formerly had been of limited potentiality. For example, the development of the Point Park and Gateway Center at the junction of the Allegheny and Monongahela Rivers in downtown Pittsburgh is considered a wise investment. This development was not undertaken before the local officials had carefully investigated the effectiveness of the flood control reservoir system that was being provided in the streams above Pittsburgh. There is danger, however, that in other localities, local officials may not be as exhaustive in their investigations when consideration is being given to rehabilitation of areas that have been blighted by floods

Some form of control of development along the streams where flood protection has been provided is therefore considered essential. As a basis for such control, it will be necessary that the entire area still subject to flooding be classified with respect to frequency and depth of flooding. Recognition will have to be given to the necessity for certain facilities and developments being at or in close proximity to the river bank. The advantages of being near the stream will have to be weighed against the probable damages that would result from such location; for example, in the case of industry, the additional cost of water pumping and additional cost of utilizing navigation facilities will have to be balanced against the potential damages to the plant as a result of flood. In effect, it will amount to establishing the types and degrees of development that can be permitted on each classification of property without increasing the frequency of excessive flood damages. For example, former waste land might be made useful for agricultural purposes or for storage of materials not subject to severe damage by flood. Areas formerly used for the latter purpose might now be made available as warehouse or mill sites. Former low-class commercial areas may become high-class commercial areas with modern stores and office buildings.

Methods of Provision

There are several methods by which flood plain zoning may be attained. It may be done by means of local ordinance, which probably would not involve payment of damages to the property owner. Without such payment, however, it is doubtful that it could be made effective in the removal of any existing developments, but might serve as a guide to the future. Another possibility would be the purchase of fee title to some areas and easements over others. A third possibility would be the general education of the public and its encouragement to follow an adopted community plan for any further developments in the area. This would at least make certain that the owner is apprised of the flood danger before he proceeds with development.

CONCLUSION

The need for some action, such as flood plain zoning, in areas that have been benefited by a limited degree of flood control, is considered essential before expansion in the flood zones and flood channels has progressed to the extent that zoning becomes impractical. This need has been at least partially recognized in some of the recent reports on flood control measures prepared by the Corps of Engineers, wherein it has been specified that zoning or some method of assuring retention of the flood discharge capacity of the stream be included as an item of local cooperation.

In closing, the writer wishes to emphasize that the opinions expressed herein are his own and not necessarily those of the Corps of Engineers.

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Discussion of "THE IMPORTANCE OF FLUVIAL MORPHOLOGY IN HYDRAULIC ENGINEERING"

by E. W. Lane
(Proc. Paper 745)

STAFFORD C. HAPP.[1]—Fluvial morphology is a field in which greater coordination between geomorphology and engineering should be mutually advantageous, and the author is to be commended for his accomplishments in furthering such coordination. His summaries and examples of engineering applications of geomorphic concepts should be helpful both to engineers and to geologists. This discussion is devoted to some differences between the engineering and geologic viewpoints, which have confused previous attempts at coordination and hence deserve attention by the author's readers.

Difference in time factors is a major problem. The geologic concept of erosion cycles involves time measured in millions of years. The average rate of stream bed lowering, in a cycle of such duration, might be of the order of one foot, or less, in a thousand years. Similar magnitudes are suggested by the amount of erosion accomplished since the earlier stages of Pleistocene glaciation. Engineering interpretations of fluvial morphology, on the other hand, are usually based upon relatively short records in which the slow changes involved in normal geologic cycles are obscured by shorter-period fluctuations resulting from normal variations in stream discharge, lateral channel shifting, cyclic or other climatic changes, or the effects of human activities. Normal shifting of sediment by stream meandering, and thick deposits of unconsolidated alluvium which fill many valleys as a result of Pleistocene glaciation, are other factors contributing to the complexity of engineering problems of fluvial morphology. The geologic concepts of youthful, mature and old age stages in stream development are therefore only of limited descriptive value from the engineering viewpoint, and do not indicate a sequence of changes within time intervals of engineering significance.

The author's classification of stream profile changes illustrates the difference between engineering and geologic viewpoints, for it omits diastrophic uplift or crustal tilting, the primary factor in erosion cycles of geologic terminology. Aside from this omission, the classification appears somewhat inconsistent by grouping into 2 classes the changes due to 3 factors of the basic equation ($Q_s d \sim Q_w s$), whereas 4 classes all depend on changes in the fourth factor of the equation. Changes in distribution, as well as volume of water discharge, may also deserve mention as factors in the classification.

It is suggested that grouping degradational and aggradational effects, but itemizing changes due to individual factors separately, would be more consistent with the form of the author's basic equation. This would facilitate brief descriptive identifications of the classes. Inclusion of the primary geologic factor of crustal changes would make the classification more complete

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from the geologic viewpoint. Such an alternative classification might be as follows:

I. Degradation, due to:
   A. Increased slope caused by diastrophic uplift or crustal tilting.
   B. Increased slope caused by lowered base level.
   C. Increased slope caused by upstream shift of base level.
   D. Increased volume or more effective distribution of water discharge.
   E. Decreased sediment quantity.
   F. Decreased sediment size.

II. Aggradation, due to:
   A. Decreased slope caused by diastrophic subsidence or crustal tilting.
   B. Decreased slope caused by rise in base level.
   C. Decreased slope caused by downstream shift of base level.
   D. Decreased volume or less effective distribution of water discharge.
   E. Increased sediment quantity.
   F. Increased sediment size.

If the revised classification here suggested does not meet with the author's approval, it would undoubtedly be helpful to many other readers if he would offer, in his closure, brief descriptive designations for his various classes of profile changes.

The premise of a "graded stream in equilibrium," on which the author's classification is based, might be improved by substitution of the term "poised" in place of "graded." "Poised" was proposed(1) for streams which satisfy the author's requirement that "the net amount of change is not sufficiently large to be detected by quantitative measurements." It is an appropriate and convenient term, lacking the connotation of confusion long associated with geologic use of the graded stream concept.

The term "graded stream" was proposed by W. M. Davis(2) to identify the condition of balance between degrading and aggrading. The suggested terminology was credited to G. K. Gilbert(3) who had previously described a stream in equilibrium as one having a supply of debris equal to its capacity, and therefore tending to establish a uniform slope by building up the gentler slopes of its bed and cutting away the steeper slopes. Davis elaborated the concept to include maintenance of an "essential balance" between degradation and aggradation throughout the long stages of maturity and old age in the erosion cycle. The apparent anomaly of long-continued degradation by a stream in essential balance between degradation and aggradation was explained by postulating that declining sediment contribution from the drainage area would permit renewal of degradation despite the concurrent slope flattening. This argument is unconvincing, however, for why should the separate processes of drainage area erosion and river bed degradation proceed at equivalent rates? If sediment production continued increasing, aggradation might continue for a long time; and if sediment production should later decline, why should this decline cease, or become slower, when the resulting degradation had just balanced the previous aggradation? Or, if sediment production should soon decline, after attaining a balanced condition, why might equilibrium not be destroyed by renewed degradation?

The difficulties in geologic usage of the term "graded stream," appear to have arisen from the unfortunate combination of two distinct concepts in the definition by Davis. One concept is that of equilibrium between aggradation

and degradation; the other is of equilibrium between the capacities of adjacent stream segments, resulting in uniformly smooth profile slopes. The latter concept involves grading in the common engineering sense, such as grading of a roadway by smoothing of abrupt irregularities or excessively steep slopes. According to this concept equilibrium must be a precise condition of balance and any deviation from such balance initiates a reaction tending to restore the balance; but it is a balance between rates of either aggradation or degradation, not a balance between aggradation and degradation.

Equilibrium between aggradation and degradation is not dependent on a precise or precarious balancing between capacity and load. Aggradation occurs if load exceeds transporting capacity; degradation occurs if erosive capacity exceeds resistance of the bed. The capacity required for degradation is greater than that required to prevent aggradation, however, because bed erosion is more difficult than transport of the resulting sediment after entrainment. Streams seldom carry a capacity load of bed sediment because they are unable to produce such material as rapidly as they can carry it away. The bed load deposits which normally occur along a stream channel do not usually indicate aggradation, but are essentially transitory deposits formed because sediment transportation is an intermittent process. The magnitude of these transitory bed load deposits depends only partly on the rate of bed erosion, and more on rates of sediment production by weathering, rain wash, creep and other mass movements, and especially stream bank erosion. If a stream, in which load and transporting capacity were just balanced, should have its capacity increased, the amount of bed load deposits would be reduced, but degradation would not necessarily follow. There would be some range of slope and volume—the controlling factors—within which the capacity would be more than adequate to prevent aggradation, yet not adequate to accomplish appreciable degradation of the bed.

Equilibrium between aggradation and degradation may persist over a comparatively wide range of slopes and volumes if a stream is flowing on hard, resistant bedrock, but the range will be much narrower if the stream bed is underlain by thick, unconsolidated sands. Engineering projects and model studies are largely concerned with the latter type of stream conditions, and perhaps for this reason engineering literature commonly treats the equilibrium between aggradation and degradation as a precarious, transitory condition. Geologic erosion concepts are concerned chiefly with uplifted mountains or plateaus in which hard, resistant bedrock is common, and hence equilibrium is a relatively more stable condition from the geologic viewpoint. The author has apparently overlooked this factor in the conclusion, stated on page 745-5, that "neither in very short nor in very long periods may natural streams be considered to be in equilibrium."

Stream gradation by development of smooth slopes, as contrasted to equilibrium between aggradation and degradation, appears to be a natural result of normal stream development. Youthful streams have excess energy due to high gradients, and erode their beds actively. Sections of the streams flowing over weak rocks are eroded deeper and reduced to lower slopes, while those sections flowing on hard rocks retain steeper slopes or even waterfalls. In this stage differential erosion tends to increase the irregularity in slopes, but the trend toward disparity in slopes is eventually reversed by accumulation of bed load materials.

The coarser and heavier bed load particles are moved only at higher stages; during falling stages they are deposited first and concentrated near

the base of the sedimentary accumulation. The larger and coarser particles are also moved less frequently and less rapidly in the flattened reaches on weak rock, and accumulate more thickly in such places. As this process continues, greater and greater floods are required to entrain the bed load accumulation and expose the underlying bedrock; and the weak rocks are subject to erosion less frequently and for shorter periods. Thus erosion of the weaker rocks will be retarded relative to that of hard rocks, and the bedrock surface will tend to become more uniform in slope. This is a process of stream gradation, by which the transporting capacity of adjacent stream segments is balanced. Such a graded stream may also attain an approximation, or at least an apparent approximation, to equilibrium between aggradation and degradation, because stream bed erosion is normally so slow. The bed load accumulation does not greatly retard lateral meandering, and a stream may effect many changes in alinement, and perhaps even widen its valley considerably during the time required for relatively little degradation.

The writer would prefer to restrict the term "graded" to a stream which had attained a smooth slope by elimination of irregularities in its profile, and which has thereby attained approximate equilibrium of transporting capacity between adjacent segments. This restricted concept would eliminate some of the difficulties which the author has recognized in his description of the equilibrium condition, but it would not be in accord with either the definition originally introduced by Davis, or the apparent intent of the author. Substitution of "poised" for "graded" would, however, free the author's classification from the ambiguities associated with the latter term. This may appear to be a minor detail, but it should be helpful in promoting clearer thinking. Those responsible for major river engineering should never forget that a poised condition may persist for a long time, yet be no indication of the effects of a flood greater than any of previous human record. Wider appreciation of this limitation of present data might even be helpful toward more adequate studies of long-term river trends, for which much of the evidence must be obtained from alluvial valley deposits.

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Discussion of "GROUND WATER PHENOMENA RELATED TO BASIN RECHARGE"

by Paul Baumann
(Proc. Paper 806)

NORMAN H. BROOKS,* J.M. ASCE.—Mr. Baumann has presented an excellent explanation of various physical phenomena encountered in ground water recharge. In this new and important phase of ground water hydraulics, Mr. Baumann has again made a significant contribution.

The writer is particularly interested in the section "Effect of Spreading Mounds on Channel Mounds." The writer concurs with the author in the basic conclusion that the recharge from a stream channel does not prevent the recharge from an adjacent spreading ground from crossing under the stream. In fact, as Mr. Baumann states, the recharge from the combination of stream channel and spreading ground is essentially the same as it would be from a stream of the same width as the combination.

However, the writer would like to point out some limitations in the author's analysis leading up to this conclusion and his corresponding equation (1). In the first place, each of the author's flow nets (Fig. 3a and 3b) shows a zone of no flow which is roughly triangular in shape in the lower central part of the previous stratum. With the usual assumption that ground water flow follows Darcy's law, there is no theoretical justification for departure from conventional flow-net analysis; hence, the center streamline should go all the way to the bottom of the pervious stratum before dividing at the stagnation point. The entire region should be included in the flow net.

It might be noted that Mr. Baumann's flow nets would certainly be correct if the wedge-shaped sections, which he has in effect excluded from the flow net, were impervious. But since they are not, there will be flow in them and the limiting streamlines cannot possibly curve downward away from the center line to the impervious base as the author has shown them.

The overall effect of this approximation may not have a very significant effect on the results inasmuch as the flow velocities in the region he has excluded are undoubtedly low. The approximation certainly makes the drawing of the flow net considerably easier.

The writer has still another misgiving about the flow nets shown in Figs. 3a and 3b. The author states on page 806-7: "Actually the flow trajectories are curved because of the sloping stream channel and aquifer, but the flow net resembles that shown in Figure 3a if developed into a plane." Theoretically, flow nets apply only for two-dimensional flow,[1] and the developing of a curved flow pattern into a plane flow net is an expedient without theoretical basis. Although developing a curved flow pattern into a plane flow net is very helpful in visualizing the flow, it is nevertheless a risky procedure for quantitative solutions of problems.

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1. It is also possible to draw a special type of flow net for axially symmetric flow in three dimensions, but there is no axial symmetry here.

Furthermore, the way in which the streamlines curve in a plan view depends upon the length of the reach of the stream from which water flows into the ground. If the recharging reach is very short, the streamlines can turn into the direction parallel to the stream quite close to the stream. If, on the other hand, the recharging reach is infinitely long, then the streamlines for the ground water flow can never become parallel to the stream.

In making the analysis which leads to equation (1), Mr. Baumann has avoided the last difficulty by assuming the underlying impervious stratum to be horizontal and the flow to be two-dimensional. However, in drawing the various flow nets which led the author to equation (1) he must have assumed some kind of downstream control. Without some control point at a finite distance, it is impossible to establish steady flow.

To make an approximate analysis of the effect of the distance to the control it will be assumed that the control consists of a horizontal discharge face. The flow net in the vicinity of this control, shown in Fig. A, is simply the well-known Kozeny solution[2] consisting of confocal parabolas. The line of seepage in this case is often called the "basic parabola." This theoretical solution is quite good except in the immediate vicinity of the spreading ground. There the curvature will be reversed because the water must flow downward initially from the spreading basin, and then turn either to the right or the left. To get an estimate of the maximum possible rate of discharge which can flow laterally through the ground toward the control, an arbitrary point P on the line of seepage in Fig. A may be assumed to be at the level of the water surface at the edge of the spreading ground.

By the properties of a parabola the length of the discharge face, a, is given by the formula

$$a = \frac{1}{2}\left(\sqrt{L^2 + D^2} - L\right),$$

where D is the height of point P above the impervious base and L is the horizontal distance from P to the discharge face. Since the discharge, q', is simply 2Ka (K is the permeability), the rate of flow can be expressed in terms of L and D as:

$$q = K\left(\sqrt{L^2 + D^2} - L\right)$$

Now the acceptance rate q as used by Baumann is equivalent to 2q/W, wherein the factor of 2 accounts for flow to either side of the spreading ground. Consequently, the maximum possible acceptance rate may be given dimensionlessly as

$$\frac{q}{K} = 2\,\frac{D}{W}\left(\sqrt{\left(\frac{L}{D}\right)^2 + 1} - \frac{L}{D}\right) \qquad (2)$$

Perhaps it would be more precise to use an inequality sign in equation (2), inasmuch as the quantity of flow represented by the right-hand side is

2. See "Seepage Through Dams" by A. Casagrande, Contributions to Soil Mechanics, 1925-40, Boston Soc. of Civil Engineers, p. 310.

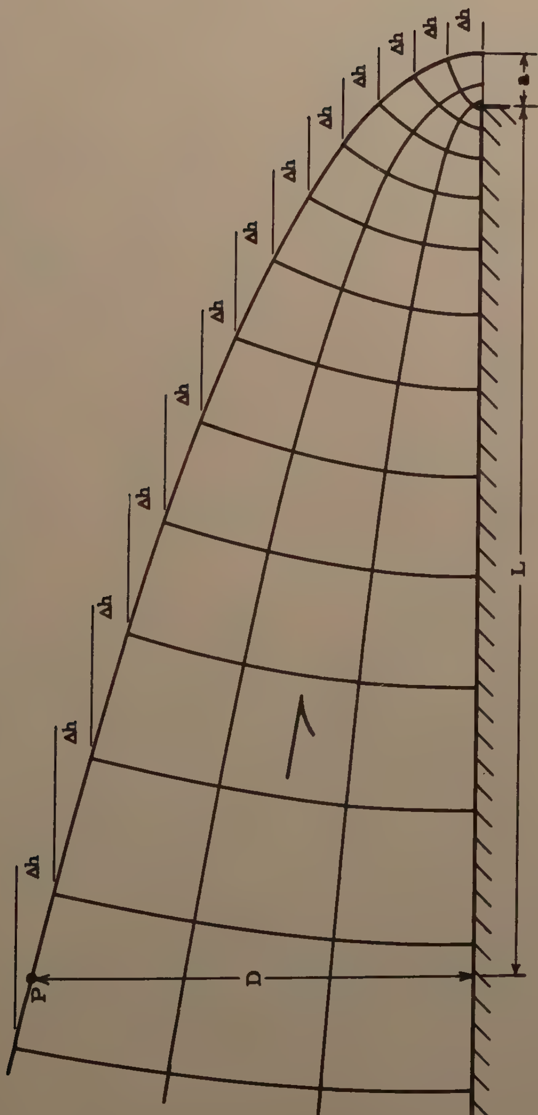

Fig. A - Kozeny's basic parabola solution for horizontal discharge face.

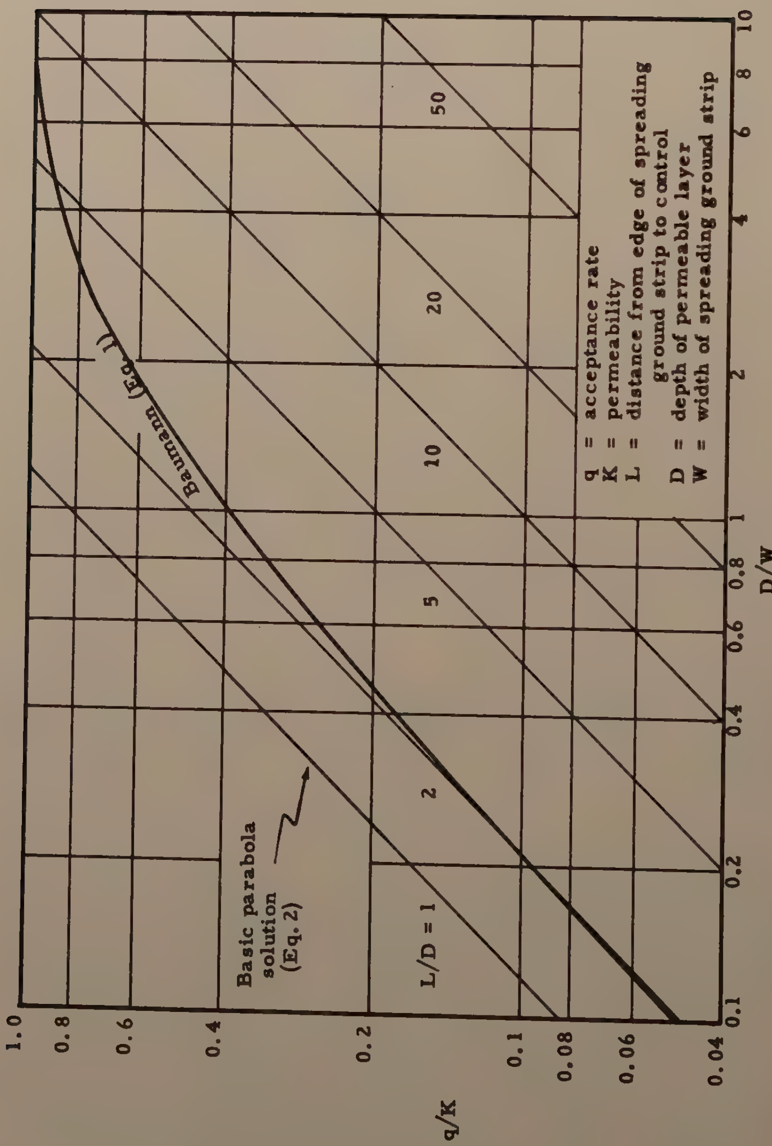

Fig. B - Comparison of Baumann formula for acceptance rate q with maximum possible q calculated from basic parabola.

undoubtedly somewhat larger than what could possibly actually occur, because the point P is taken to be at the same height (D) as the top of the spreading ground. This condition is impossible because of the initial downward flow, but the use of D still gives an upper bound for the lateral rate of flow and hence the acceptance rate for the spreading ground.

In Figure B a comparison is made between the author's equation (1) and the writer's equation (2). It is apparent that for many values of L/D the author's equation (1) gives a value of the acceptance rate which exceeds the capacity of the pervious stratum for transmitting the flow horizontally to the control. For example, if $L/D = 10$ and $D/W = 2$, then the upper bound by Eq. (2) is $q/K = 0.20$; on the other hand, the author's Eq. (1) gives $q/K = 0.63$, a value which is at least three times too large for this case. In fact, the distance to the control (L) must not be more than about twice the depth (D) if the value of q found by the author's equation (1) is to be consistently less than the maximum possible as indicated by equation (2). It appears then that the distance from the spreading ground to the control is a very important variable in the problem which must be taken into account.

It may be noted from equation (2) that for some combinations of variables, the calculated value of $\frac{q}{K}$ will exceed 1. This is physically impossible, because the acceptance rate can never exceed K, which is the rate corresponding to downward percolation without hydrostatic pressure. Consequently, in these cases where $q/K > 1$ by Eq. (2), the recharge mound cannot possibly rise up to the spreading ground level, but reaches its ultimate height some distance below the spreading ground, as pointed out by Mr. Baumann on page 806-3.

FREDERICK L. HOTES,[1] M. ASCE.—Ground water problems of all types are increasing throughout the United States. Although there are many complex phenomena connected therewith that engineers and hydrologists would like to know more about, none are more important than that of basin recharge. Recharge, both natural and artificial, places definite limits on the safe yield of ground water basins on a sustained basis. The author has set forth some of the phenomena connected with recharge problems for both free and confined aquifers, and has indicated some of the practical limitations learned from personal experience.

This discussion will be limited primarily to topics associated with sea water intrusion into pressure aquifers. Several research projects conducted at the University of California, Berkeley, California, during recent years have contributed information of considerable assistance in the interpretation of ground water behavior in coastal regions, and on the effectiveness of injection wells in repelling sea water intrusion.

Model Studies

In addition to the contract with the Los Angeles Flood Control District mentioned by the author, the California State Water Resources Board also contracted with the University of California to conduct laboratory and model studies of sea water intrusion. It was intended that these studies would supplement and aid in the prototype experiments being performed by the Flood Control District.

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Model Aquifer

Following a thorough search of the literature,[2] a sand and lucite model was constructed. A detailed description of the model and the experiments, together with the results of research on impermeable cutoff wall materials, has been previously published,[3] and hence only a summary will be presented here. Figure 1 is a schematic sketch of the apparatus.

The main chamber, representing an idealized pressure aquifer was 4 feet long, 6 inches high, and 3 inches wide. It was packed with a commercially available washed quartz sand, Del Monte 30 mesh. This sand has an effective size (10 percent) of 0.22 mm, a 50 percent size of 0.28 mm, and a uniformity coefficient (60/10) of 1.3. When packed within the model its coefficient of permeability averaged 0.032 cm/sec (680 gallons/square foot/day). The salt water was introduced through the chamber on the left and the fresh water through the chamber on the right. Both fluids were supplied through constant head flow regulators.

The piezometric tubes were used to establish the piezometric surface under any desired condition flow. The model injection wells were located only along the near side of the aquifer. They were made of split glass tubing one fourth inch in diameter. The near face represents a plane of symmetry, normal to the coast line, and through the centerline of one of a line of injection wells of equal spacing parallel to the coast. The far face represents the plane of symmetry normal to the coast and midway between two injection wells. No flow crosses these planes of symmetry in a uniform injection process, so they are represented by solid lucite walls in the model.

Model Fluids

The salt water used in the model was adjusted to a specific gravity of 1.100. Both calcium chloride and sodium chloride solutions were successfully used. This procedure increased the model flow rate and generally speeded up an otherwise very slow action. The viscosity of these solutions was about 30 percent greater than that of sea water. This discrepancy did not affect measurements when the sea water wedge was stationary, but did have some effect on rates of wedge movement.

A novel idea that produced startling and successful results was the use of fluorescent dyes to aid in distinguishing the fresh from the salt water. A convenient concentration of ordinary dyes does not show up well in the model because of the thin layer of solution visible between the transparent sides and the first layer of sand grains. An ultraviolet light source was also used to produce a strong, positive response from the thin solution layer next to the wall. Rhodamine B in the saline water and Fluorosol in the fresh water gave good contrasting colors of red sea water and blue fresh water.

Results

Harder,[3] by an adaption of Muskat's[4] approach to the solution of seepage through an earth dam on an impervious foundation, derived the following expression for the fresh water flow and length of intruded saline wedge under equilibrium conditions.

$$q = (1/2)(S-1)\,\frac{M}{L}\,T \qquad \text{Eq. (1)}$$

in which

- q = seaward fresh water flow per foot of ocean front
- S = specific gravity of sea water
- M = thickness of aquifer, down to the lowest depth which must be protected
- T = aquifer transmissibility for a 100 percent hydraulic gradient
- L = length of sea water wedge, from ocean outlet to the toe

It is to be noted that the flow is independent of the depth of the aquifer below sea level.

Figure 2 shows the basic relationships involved and the confirmation of the equation by experimental results, shown plotted as circles. Fresh water is flowing from right to left. The same equation was obtained from potential theory by assuming a parabolic interface between fresh and salt water. It applies only to an idealized aquifer that would never be found under natural conditions. However, when applied with good engineering judgment it can be quite useful in the study and solution of practical seawater intrusion problems.

Figure 3 is a composite photograph showing the postion of the wedge for three different values of fresh water flow. The dark vertical lines are caused by the metal bracing of the model. The shapes of the interface are typical. The distances shown are for a hypothetical prototype aquifer 100 ft thick, but are not limited to this one case. One of the great advantages of a model such as this is that the results can be extended by means of model laws to any prototype aquifer. A new model does not have to be constructed for each aquifer—the single study serves all. Of course, models of aquifers with known variations in permeability might justify individual models for the study of peculiar conditions.

Figure 4 shows the behavior of the sea water wedge when water is injected through a line of wells 1000 feet inland, and spaced at 500 foot intervals. In this series there is no flow to or from the inland area. Hence all the injected water escapes to sea, and resists the intrusion. In the series on the left the injection rate is such that it would be sufficient to hold the wedge steady at 2000 feet. Since the flow is not sufficient to hold the wedge seaward from the well, however, sea water moves between the wells and enters the inland region as a density current. On the series on the right the injection rate has been increased threefold. This rate is sufficient to hold the wedge seaward from the well at about 667 feet from the ocean outlet, so that part of the sea water wedge inland from the well is completely cut off.

An approximation of the prototype time involved in these photographs will indicate the value of the model in speeding up an inherently slow process. The Silverado aquifer is approximately 100 feet thick. Its permeability as obtained from an informal conversation with the author is 25 feet per hour (4500 gallons/square foot/day). The prototype—model time scale ratio is therefore approximately 2900 to 1; or, one minute of model time represents 2900 prototype minutes (approximately two days). The time after start of injection for each photograph in Figure 4 is as follows.

Frame No.	Left	Right
1	0	(Before injection)
2	6 months	19 days
3	2.5 years	4.7 months
4	5.5 years	3.5 years
5	8.1 years	10.8 years

Figure 5 more nearly represents conditions at the Manhattan Beach test site described by the author. The series of Figure 5 (a) shows the sea water wedge intruding from left to right under an inland overdraft condition. Fresh water is being withdrawn from the right at a prototype rate of 900 gallons/day/ft. It should be noted that the wedge interface is no longer concave downward, but, once inland flow is established, closely approximates a straight line, with localized variations at top and bottom. The interface lengthens as it advances inland. The toe velocity is rapid at first, then gradually decreases, and asymptotically approaches the average velocity determined by the rate of withdrawal and the effective cross sectional area.

This straight line interface between two fluids with different densities and moving through porous media has also been demonstrated by Naor[5] in a Hele-Shaw model. Naor hypothesized that the interface is actually the result of two superimposed linear motions. One is the horizontal movement of a fluid of single density and gives a vertical front. The other motion is that investigated and described by Keulegan.[6] Keulegan observed the motion of the interface between two fluids of different densities in a porous media originally separated by a vertical gate. Upon removal of the gate a tilting motion occurred. The interface appeared to rotate about the mid-height, and to remain a straight line during the motion. The superposition of this linear rotation on the linear horizontal displacement results in the straight line interface observed by Naor and seen in Figure 5 (a).

Figure 5 (b) shows the effect of injection through a well located 2000 feet inland at a prototype rate of 415 gpm for wells spaced at 500 foot intervals. This rate is sufficient to provide for the overdraft of 900 gals/day/ft, and the seaward flow of 293 gals/day/ft required to stabilize the wedge at 2000 feet inland from the coast.

It should be emphasized at this point that the toe of the wedge would probably be about 2250 feet inland midway between the wells. The result of this tendency of the wedge to work inland between wells is clearly seen in the author's Figure 8. It is recommended that the line of injection wells be located at least one half well spacing inland from the allowed maximum intrusion of the wedge. The flow from the injection wells tend to equalize at this distance from the wells and a relatively uniform flow rate per foot of coast is obtained. If the wells remain at 2000 feet, a greater injection rate is required to keep the wedge at the midpoint between wells from passing beyond the line of wells. This means greater loss of injected water, at no benefit.

One of the most important results of the test was to show that the wedge would be stabilized at the same point, for the same rate of seaward flow, no matter how far the wells were spaced inland, as long as they were at least one half well spacing beyond the maximum allowable intrusion distance. Tests were also made on wells penetrating only the upper and/or the lower half of the aquifer, and from distant inland areas. All results confirmed theory—the location of the source of the seaward flow was immaterial, within the limits previously mentioned.

The distortion of the vertical scale is tenfold in Figure 5 as compared with the Silverado aquifer. This vertical exaggeration should give a fair indication of the relative importance of the diffusion zone at the interface. The interface between the sea water wedge and the overlying fresh water remained quite sharp in the model and such should be the case for the prototype. Even the fresh water cone from the injection well showed little mixing with the fresh water originally in the aquifer, but tended to move out by a replacement process.

In addition to the still photographs, a 16 mm technicolor motion picture, with titles, was made by means of lapse time photography. This film is available on loan from either the California State Water Resources Board or the University of California.

Injection Wells for Fresh Water Barrier

The use of injection wells for the formation of a "fresh water barrier" against sea water intrusion is a tempting concept; and perhaps a practical and economical alleviating solution to the problem in some cases. However, certain basic principles must be born in mind if this method is to be the most economical solution. There are four broad questions to be answered.

1) How much water should be injected?
2) Where and how should the wells be located?
3) How can the required amount be injected into the aquifer?
4) Is this the most economical solution?

How Much Water?

The answer to this question lies in the principles of the Conservation of Matter, and of Continuity. Sea water intrusion is caused by an inland overdraft condition. The laws of nature require that a small amount of fresh water continually flow out to sea to repel the sea water. This amount can be estimated from equation (1) for any specified allowable location of the toe of the wedge. Whenever this seaward flow is reduced the wedge will move inland. If the flow is not only reduced, but eliminated, and perhaps even made a negative quantity by inland pumping, sea water will flow inland. Wedge stability can be achieved only by furnishing a seaward flow of fresh water. Water in sufficient quantity will not flow seaward unless the overdraft causing the invasion is eliminated. In simple language, <u>the total amount of water required is equal to the sum of the seaward stability flow plus the existing overdraft</u>. This amount of fresh water can be supplied by several methods or combinations that will be discussed later.

Location of Wells

The writer is not as optimistic as the author regarding the status of the sea water trapped inland from the line of injection wells, and shown in Figure 6 (b) of the author. The writer believes that this salt water will continue to move inland and will begin to appear in wells that have previously been in a fresh water zone only. Figure 8 of the author indicates that this is exactly what is taking place at Manhattan Beach. The iso-chlors of Figure 5 that are inland from the injection wells are even further inland in Figure 8. Any wells pumping near the wedge will induce some of the salt water to flow to them. This salt water will mix in the well with fresh water. It cannot be removed except by pumping, as long as the injection wells remain as shown.

The writer recommends that injection wells to be used for sea water repulsion <u>be located inland from the most advanced intrusion of the wedge toe.</u> The same amount of injected water is required. This location has the great advantage of immediately causing <u>all</u> the wedge to begin a retreat to the sea. All pumping wells should improve in quality and none should become more saline. All the invaded sea water will be flushed out into the ocean and the aquifer will once again be fresh.

Problems of Injection

It is well recognized that it is much more difficult to get water into the ground by pumping than out of it. The large scale tests reported by the author are certainly some of the most significant yet undertaken. Experiments conducted on a well field (Figure 6) at the University of California have yielded interesting results on injection rates for both clear water and water containing 20 percent of primary settled sewage.[7, 8] Clear water injection rates of 8.4 gallons per foot of aquifer with a permeability of 1900 gallons per square foot per day were successfully obtained. The sewage injection trials were also successful, but a redevelopment technique, reported in detail in the references, is vital. Of especial importance was the finding that no coliform organism appeared in the observation well some 190 feet away. Some coliform organisms did appear at a distance of 100 feet. Increasing the injection rate beyond a certain limiting value did not increase the travel distance of coliform organisms. These studies confirmed again the inherent natural filtering action of an aquifer. If sewage rather than domestic water can be used for recharge by injection, considerable savings in cost can possibly be realized. Further studies of this aspect are believed warranted.

Economical Solutions

The quantity of water required to repel sea water intrusion can be estimated. Usually the total water use in a critical area cannot be expected to decrease, but on the contrary, present estimates usually predict even greater water use in such areas. One part of the solution is obviously then either additional importation of water, reclamation of waste water, or both. Once this supplemental supply is brought into a overdraft region, its disposition should become a subject of economic analysis. The sea water intrusion can be eliminated by either of the following methods.

1) Injecting the water into the aquifer.
2) Supplying the water directly to the users through a surface distribution system. This reduces the draft on the aquifer and allows the required seaward flow to take place.

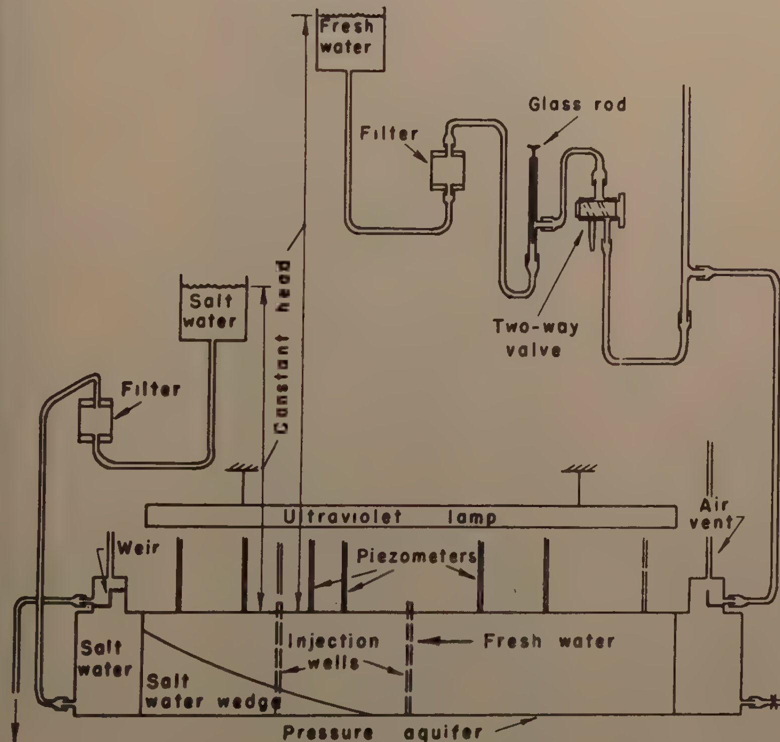
The total amount required is the same in each case. However, Method One requires the water to be pumped in, and then pumped out again. Obviously this costs money, and probably should not be justified unless this is a cheaper method than the construction of a surface distribution system. Thus the value of the pressure aquifer is as a conduit. It provides no storage, but provides a "spigot (well)" for each overlying land owner.

In the case of an unconfined aquifer, there are additional benefits to be credited to Method One by its keeping sea water out of the reservoir during a period of dry years, when the water users may lower the ground water level below sea level.

The value of the aquifer in acting as a purifying filter for injected sewage must also be considered in cases where waste water is available in sufficient quantities.

Ground water problems are inherently more difficult to grasp than surface water problems, because in most instances they cannot be actually seen, and must be dealt with in abstract terms, mathematics and drawings. It is only through studies and tests such as the author has described, that can practical solutions be found. His contributions to the literature are appreciated.

SCHEMATIC SKETCH
of
APPARATUS



Note This apparatus can be arranged for different kinds of tests by simply inserting the fresh water and/or the salt water and/or the effluent at appropriate places on the model The above set-up shows fresh water being recharged from an inland area.

Figure 1.

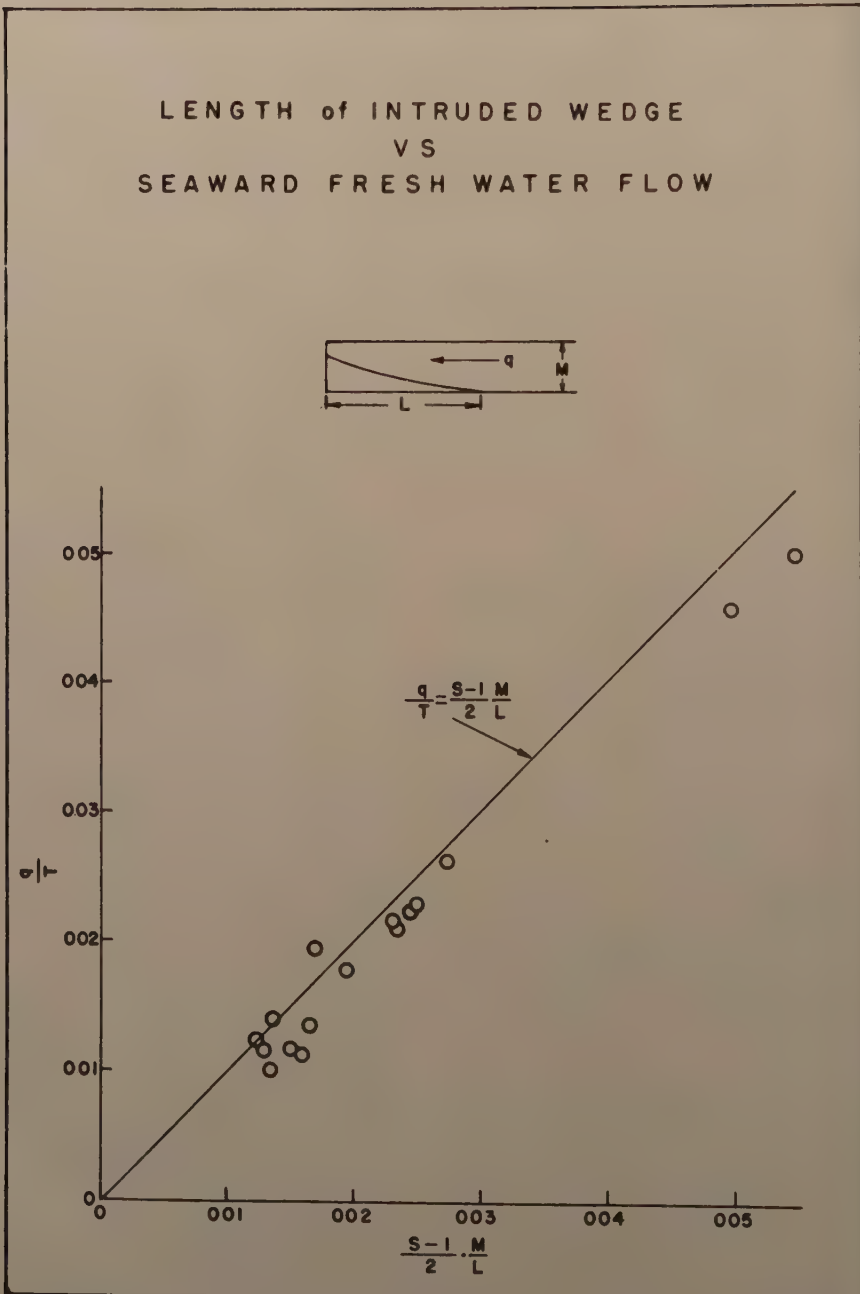
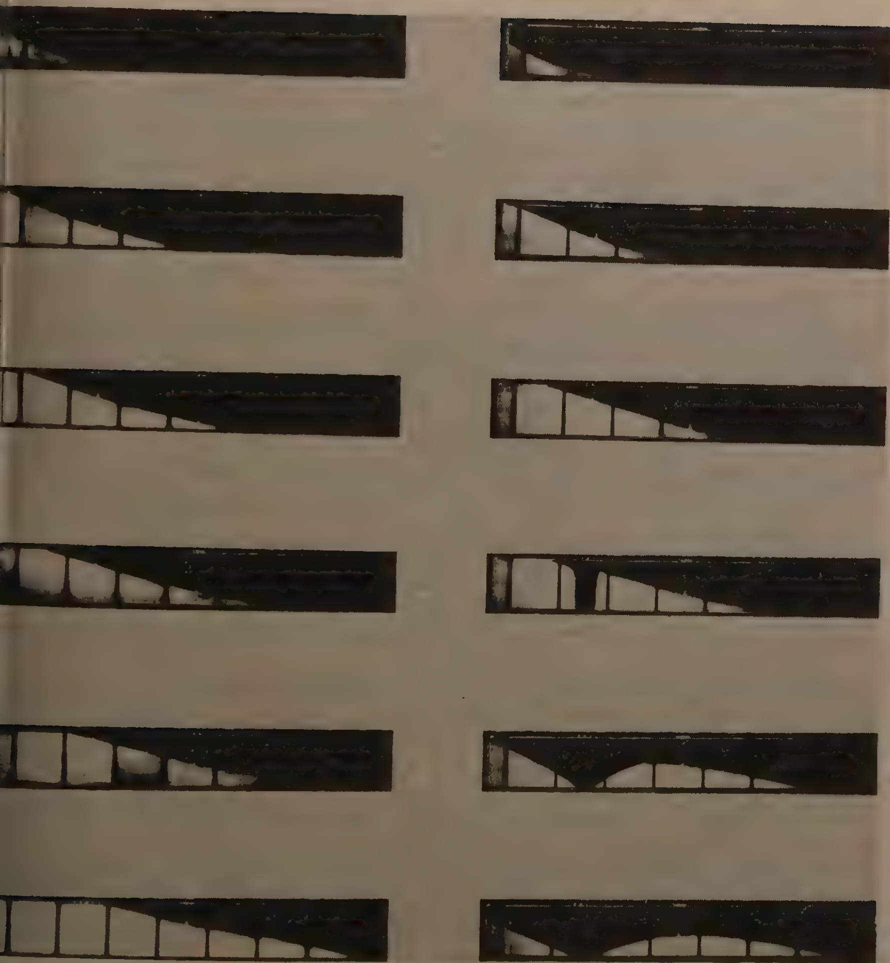

LENGTH of INTRUDED WEDGE
VS
SEAWARD FRESH WATER FLOW
q
M
L
$\frac{q}{T} = \frac{S-1}{2}\frac{M}{L}$
0.05
0.04
0.03
0.02
0.01
0
$\frac{q}{T}$
0.01
0.02
0.03
0.04
0.05
$\frac{S-1}{2} \cdot \frac{M}{L}$

Figure 2.

Figure 3. Shape And Position of Sea Water Wedge At Three Different Seaward Flow Rates



re 5. Behavior of Sea Water Wedge Under Overdraft Conditions. With and Without Injection of Fresh Water

(a) (b)

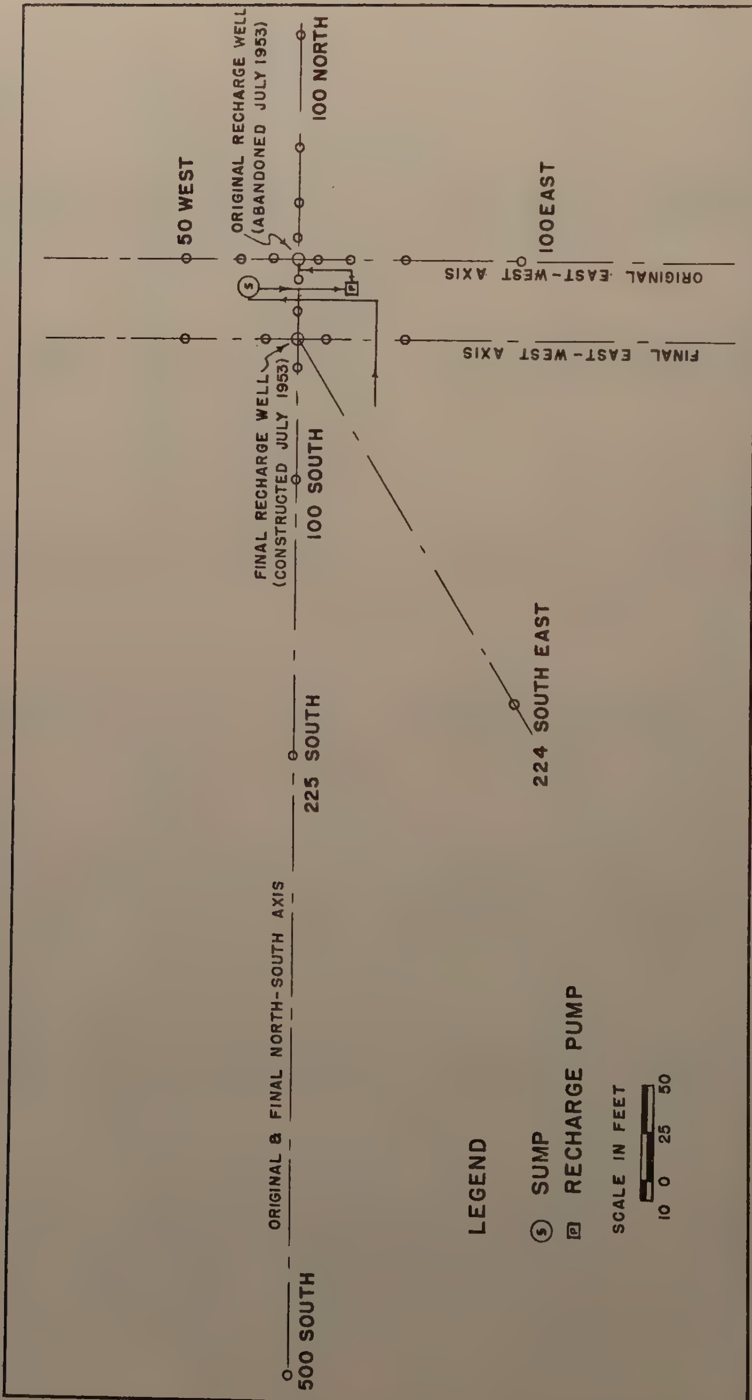


Figure 6. Layout of Well Field

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4. "Flow of Homogeneous Fluids Through Porous Media," by M. Muskat, McGraw-Hill Co. 1937.

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JAMES A. HARDER,[1]—Mr. Baumann's description of the ground water recharge operations being undertaken by the Los Angeles County Flood Control District is a welcome contribution to the literature dealing with water conservation—a subject not only of present vital importance to such semi-arid regions as Southern California, but one sure to be of increasing importance in other parts of the United States. This writer would beg to take exception, however, to certain of his statements minimizing the importance of the "saline wave."

As shown by the iso-chlors in Figure 5 of the paper, at the time injection was begun the injection site was underlain by water having a chloride ion concentration about 90% of that of sea water. This fact lead to a fear that if this particular site were to be used, the sea water thus trapped would cause trouble as it moved inland. Accordingly, the State of California (which provided the initial financial support) required the District either to use a site inland from the intrusion or to obtain waivers from major water users exempting the state from responsibility for damage done. The waivers were obtained, and the program proceeded.

Returning to the iso-chlors drawn in Figure 5 of the paper, an estimate may be made from them of the total chloride content of the water which lay behind the injection well site before injection. This can be done by making a profile of the concentration vs. distance inland from the injection well site and applying graphical integration. Based on profiles through the end points and the center of the site, and assuming an aquifer thickness of 100 feet and a porosity of 30%, the average quantity of chloride ion existing behind the

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4,000 foot long site is thus computed as being about 20 tons of Cl^- per foot of front, or about 80,000 tons altogether. This amount of Cl^-, by simple arithmetic, can contaminate 240,000 acre feet of water to a concentration of 250 parts per million, a generally accepted maximum for domestic use.

Of course there is a large amount of fresh water yet remaining in the inland portion of the aquifer which might be available for dilution of the saline wave. From Figure 4 of the paper the distance from the injection well site to the Inglewood Fault, the inland boundary of the basin, is about six miles. Assuming that the aquifer thickens to 150 feet and continues to the fault, there are 150,000 acre feet of fresh water within the 4,000 foot wide strip between the injection well site and the fault. The above computed weight of chloride could contaminate this water to a concentration of nearly 400 parts per million if it were uniformly distributed. All of this fresh water will not be available for dilution-duty, however, as inland water users may be expected to liberate as much as possible of it from the burden of acting as a diluting agent.

Furthermore it does not appear that the natural recharge will contribute sufficient fresh water to be significant in the diluting effort. On the basis of its area relative to the basin, this strip behind the injection site should expect to receive no more than 2,000 acre feet per year of new water from natural recharge. Artificial recharge from the injection well site is contributing a like amount, from the authors data.

There remains the hope that the fresh water which overlies saline water in the "wave" or "trapped in depressions along the bottom of the aquifer" could still be recovered free of salt by means of partially penetrating wells. This is possible, but only under severe restrictions on the average pumping rate. For example, a calculation based on experimental data from Muskat[2] will show the maximum drawdown at which a well penetrating the top fifty percent of the fresh water zone of an aquifer can operate without "coning." Assuming that there is a layer of sea water lying at the bottom of a 100 foot aquifer, a drawdown of only 3.5 feet will induce the salt water to rise in a "cone" and enter the bottom of the well. Obviously this is a much smaller drawdown than is usually encountered in pumping wells supplying a normal yield.

An idea of the past, present, and future status of sea water intrusion in Manhattan Beach may be had from Figure 5(b) of the discussion by Professor Hotes in this issue. This is a series of photographs taken by the writer of sea water intrusion into a model aquifer constructed at the University of California, Berkeley.[3] Overdraft and injection rates were set up to approximate conditions at Manhattan Beach; and with scale factors adjusted so that the four foot long model represents a section of the aquifer 8,000 feet long perpendicular to the coast, the total time elapsed between the first and last frames represents ten years. The present situation would be similar to a frame between numbers four and five. In frames five and six the saline wave is shown moving inland together with the fresh water from the injection wells at a rate corresponding to about 500 feet per year in the prototype. The shift

2. Muskat, Flow of Homogeneous Fluids Through Porous Media, first edition, pp 480-496, McGraw-Hill, 1937.
3. Report on Laboratory and Model Studies of Sea Water Intrusion, Technical Bulletin 11, Sanitary Engineering Research Laboratory, University of California, Berkeley 4, California.

in the iso-chlors between Figures 5 and 8 in Mr. Baumann's paper seem to indicate the same approximate rate. The wave tends to flatten with the passage of time.

CHARLES H. LEE,[1] M. ASCE.—A very interesting equation is presented in this paper (Equation 1) expressing the relation between the average acceptance (infiltration) rate per unit of wetted area and the parameter $\frac{D}{W}$ where D is the depth of aquifer to impervious stratum and W the width of stream channel (or stream channel plus spreading ground where the latter is adjacent to the stream). The author well points out that this parameter is of economic importance. The writer also believes that it is of physical significance. In order to illustrate this graphically the writer has plotted the full range of $\frac{D}{W}$ for K = 1000 Meinzer Units (gals. per sq. ft. per 24 hours for gradient of unity (Writer's Diag. 1). This value is equivalent to a flow of 67.5 cubic feet per second per acre for gradient of unity. The curve approaches closely to its asymptote, $q = 67.5$, for values of $\frac{D}{W}$ exceeding 8. The depth of unconfined water-bearing alluvial deposits as found in nature seldom exceed 500 ft. and is ordinarily much less. Spreading basins of 500 to 1500 ft. width, and more, are in common use. For a basin of only 500-ft. width the maximum value of $\frac{D}{W}$ in actual practice would, therefore, seldom exceed unity. It is usually very much smaller. For $\frac{D}{W} = 0.1$ the value of q is 3.6 sec. ft. for gradient unity, as indicated by writer's diagram 1.

Referring to the model tests which the author has performed to supplement his mathematical analysis it is found that the value of the parameter $\frac{D}{W}$ in the model was $\frac{12.8 \text{ ins.}}{1.2 \text{ ins.}} = 10.7$.[2] This extreme divergence in value of the parameter $\frac{D}{W}$ between model and prototype leads one to question the propriety of relating model and prototype as does the author (p. 806-4). Certainly in the second phase of mound growth, where the surface of the mound contacts water in the spreading basin and acquires the potential control of its surface, there is far greater probability of "water logging" of the surrounding ground for $\frac{D}{W} < 1$ that for $\frac{D}{W} = 10$. Where the spreading basin is adjacent to stream channel there is also a greater probability of substantial return flow from spreading basin to the stream channel.

Commenting on the latter possibility the author states that "no significant return flow from the spreading basin to the stream channel will materialize so long as the gradient between the two is moderate," (p. 806-8), suggesting a slope of 5 horizontal to 1 vertical as critical but not supporting this suggestion by theory or test. The only positive statement the author has made in this connection is that "to stop the flow from the spreading basin the water

1. Cons. Engr., San Francisco, Calif.
2. Baumann, Paul, Ground water movement controlled through spreading, Trans. A.S.C.E., 1952, Vol. 117, Figs. 18 to 21.

surface of the channel mound would have to rise to the same elevation as that in the spreading basin." Author's Fig. 3b is referred to in illustrating this statement. The writer has attempted a sketch development of the flow net in the region of an assumed river levee (writer's Diag. 2). This is more detailed than is shown on the author's Fig. 3 and is one which the writer considers to be more in accord with actual field experience as well as with results of model tests for percolating flow under levees and dams. The percolating flow through the vertical section A-A', immediately back of the near river bank in a slice 1 ft. wide perpendicular to the river bank, can be computed from the equation $Q = KAi$, assuming K as 1000 Meinzer units, A = 1 ft. x 92 ft. = 92 sq. ft. and $i = \frac{6'}{37'}$

$$Q_A = \frac{1000 \times (92)}{7.5 \times 24 \times 60 \times 60} \times \frac{6}{37} = 0.32 \text{ sec. ft.}$$

A similar computation at the far bank of river in vertical section B-B' is

$$Q_B = \frac{1000 \times 92}{7.5 \times 24 \times 60 \times 60} \times \frac{6'}{230'} = .023 \text{ sec. ft.}$$

The indicated loss must be assumed to be seepage into the river channel, $Q_A - Q_B = .286$ sec. ft.

This is $\frac{.286}{.32} = 87\%$ of the percolating flow at the near bank of the river, which percentage would be greater for sections taken at lesser angle to the river. This computation illustrates that even for a gradient flatter than 5 to 1, the seepage into the river may be "significant." The author's optimism regarding avoidance of significant return flow from spreading basin to channel at gradients less than 5 to 1 would appear to be unfounded in the case of actual field conditions.

Broadly viewing the author's mathematical approach, as developed in his earlier paper, it may be pointed out that there is no law of physics but which is true only within certain limits. The author in developing his equations has omitted to investigate and define limits and in his former paper[2] does not indicate within what ranges his physical phenomena are mathematically valid. For example, in setting up boundary conditions for unsteady flow he states in condition No. 2 that $y = 0$ at $x = L_d$ for all values of t in which $0 < L_d < \infty$. This disregards the case where seepage escapes from the bank of a ditch, river, or lake above free water surface, or the existence of escape gradient. It also assumes that the advancing margin of the mound has a feather edge rather than blunt nose. The rapid initial rise of water, as often observed in test holes following initial stream flow, would indicate a blunt nose.

In the author's condition No. 3 he states $y = f(x)$ at $t = \infty$, apparently assuming that the profile of the water table is a continuous linear function of distance from spreading ground throughout its length at any and all times. This assumption is predicated upon the transmitting material being homogeneous for any value of x. This is a condition never met with in nature. It also omits consideration of discontinuity due to intervening horizontal water surface in open ditch or river. Such condition is of common occurrence.

The "fitting" of the "basic parabola" observed in model tests with the

2. Op. cit., Equation 4, page 1027.

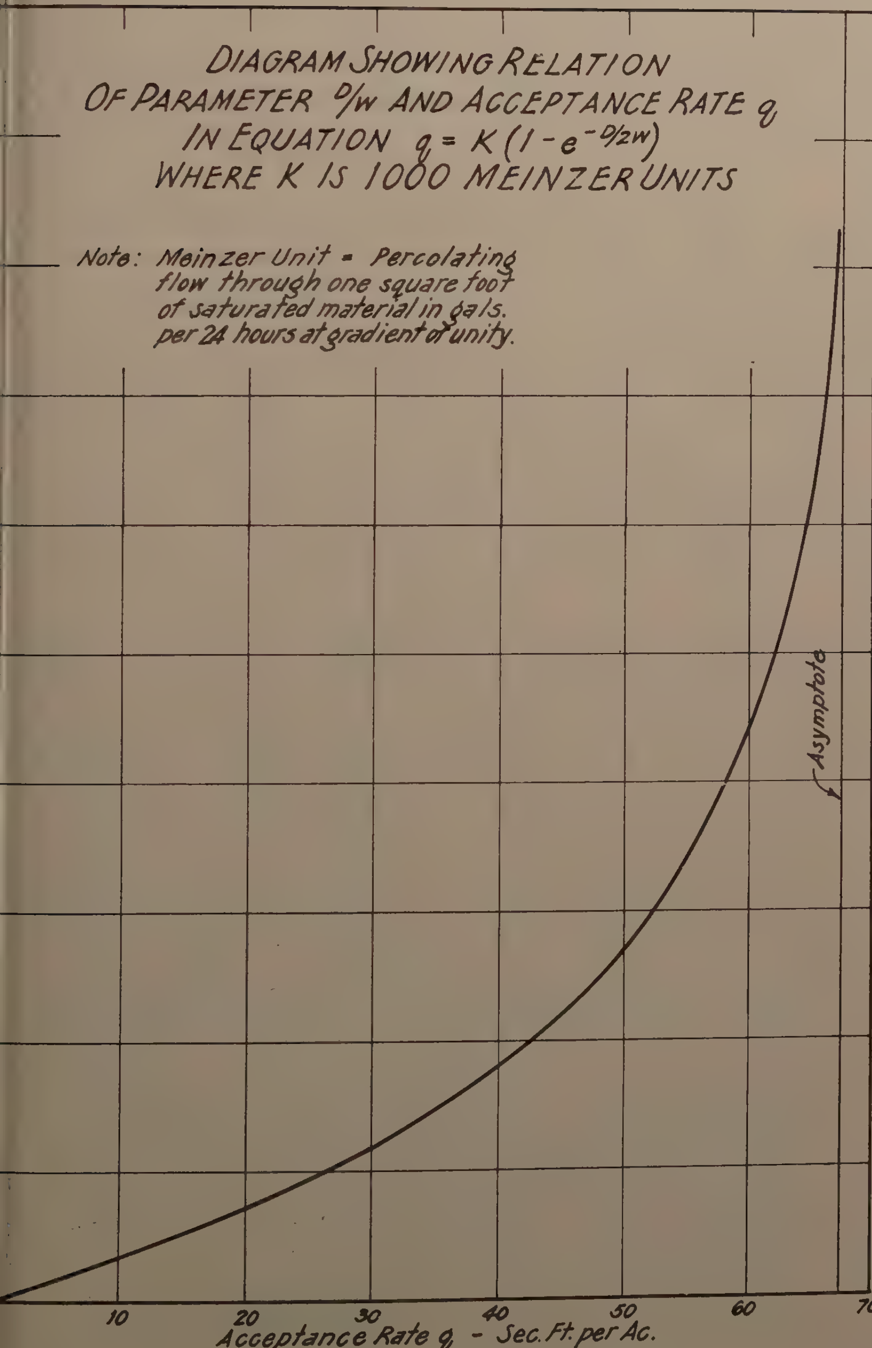
DIAGRAM SHOWING RELATION
OF PARAMETER D/W AND ACCEPTANCE RATE q
IN EQUATION $q = K(1 - e^{-D/2w})$
WHERE K IS 1000 MEINZER UNITS
Note: Meinzer Unit = Percolating flow through one square foot of saturated material in gals. per 24 hours at gradient of unity.
Asymptote
10
20
30
40
50
60
70
Acceptance Rate q - Sec. Ft. per Ac.

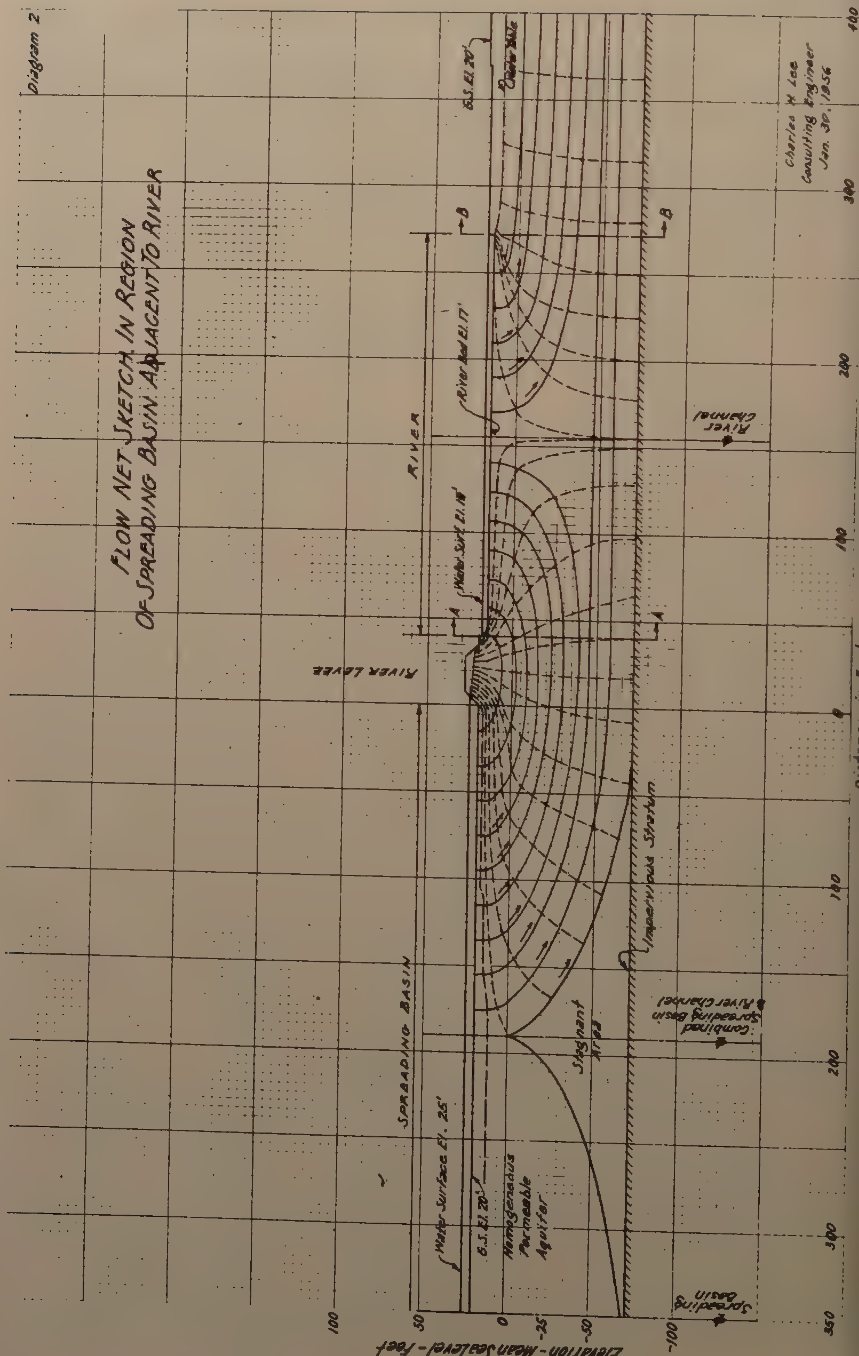
Diagram 2
FLOW NET SKETCH IN REGION
OF SPREADING BASIN ADJACENT TO RIVER
Charles H Lee
Consulting Engineer
Jan. 30, 1956
RIVER
RIVER LEVEE
SPREADING BASIN
River Bed El. 17'
Water Surf. El. 18'
G.S. El. 20'
Water Surface El. 25'
G.S. El. 20'
Homogeneous
Permeable
Aquifer
Stagnant
Area
Impervious Stratum
River Channel
Combined
Spreading Basin
& River Channel
Spreading
Basin
Distance in Feet
Elevation - Mean Sealevel - Feet
100
50
0
-25
-50
-100
400
300
200
100
0
100
200
300
350

arabolic type of equation developed mathematically for the mound profile for teady flow does not justify the extension of such equations beyond the specific imits of the model test. Obviously the results of such tests apply only to the onditions established by the model. In the absence of a great variety of conitions, which only a great number of tests made on many different models ould provide, there exists no foundation for generalization or correlation ith mathematical equations.

The author points out that his basic equation for unsteady flow[3] has its nalogue in the flow of heat through a prismatic non-radiating bar. It is to e noted that for this condition to obtain for percolating flow through granular oil, the soil stratum must be enclosed in sealed surfaces which, in a flow et, would be represented by flow lines rather than by equipotential lines. In ractice, this condition is fulfilled for a percolating stream in homogeneous nconfined granular by the free water table and the impervious stratum at the ottom. When such a flow is intercepted by an open ditch, river or lake, howver, the upper limiting surface changes from the water table to the stream ank and bed. The stream bank and bed then becomes an equipotential line, hus changing the analogue to heat transference through a radiating prismatic ar. Hence, at this point a discontinuity occurs in the mathematical equations elating to the percolating stream. Recognition of this condition is an aid in nderstanding why a substantial part of the percolating water seeps into the tream channel rather than moves "across the channel," as stated by the uthor (p. 806-7).

ROBERT T. KNAPP,* M. ASCE.—Mr. Baumann is to be congratulated on his excellent paper on a very important subject. Although it was undoubtedly timulated by his interest in the water problems of Southern California, the oncepts and methods it presents are applicable to all semi-arid and arid egions where water supply is the limiting factor in determining the size of opulation that can be supported by the area.

The writer was particularly interested in the Manhattan Beach experiments n the development of a fresh water barrier to prevent salt water intrusion in epleted groundwater basins. While following the action of the fresh water arrier as shown in the author's Fig. 6a, b, and c, the writer observed a inor discrepancy in Fig. 6c which seemed to have some interesting impliations. The author states that Fig. 6c was drawn with the aid of a crystal all. This figure represents the idealized ultimate condition when equilibrium ecomes established. It seems to the writer that when the author was gazing rough the crystal ball at the ultimate position of the free surface of the ndward saline wedge, there must have been a minor irregularity in the rystal, because the free surface of the salt water appears to slope in the rong direction.

It seems reasonable to assume that in a density current flow in a porous edium, there will be a vanishingly small friction at the interface between e two liquids. In the first place, the relative velocity will be extremely mall. In the second place, the porous medium is fixed in space, and the hear that restricts the flow of the overlying layer is that between the liquid this layer and the porous medium. Hence, in the case of salt water inusion, it can be assumed that, for equilibrium conditions, pressures will

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Op. cit., Equation 4, page 1027.

Prof., Hydr. Engr., California Inst. of Technology, Pasadena, Calif.

always be constant along any horizontal plane and will be equal to the weight of a unit column of sea water whose height is the distance from that horizontal plane to sea level. The slope of the interface will therefore be determined solely by the pressure from the overlying layer.

In the case of the landward saline wedge cut off from the ocean by the recharging flow, pressures must also be constant along horizontal planes, but the pressure level will not be determined by the depth below the surface of the ocean, but solely by the pressure in the overlying fresh water. The configuration of the interface between this isolated body of salt water when it finally reaches equilibrium will depend upon its geographical location at equilibrium. It will, of course, run downhill to the lowest available spot and there form a lake. In Fig. 6c it looks as if this lake was formed within the recharge zone, that is, where the fresh water mound is still sloping landward If this is the case, the interface would slope seaward, as shown in the writer' Fig. 1. If this salt water lake should happen to come to rest under a region in which there were pumping wells, then the interface would include salt water mounds whose peaks would lie under the pumping wells, as shown in the writer's Fig. 2.

This principle of zero shear at the interface makes it very easy to calculate the possible distance of the salt water intrusion from the known groundwater elevations since the hydrostatic pressure at the interface must just equal the depth below sea level of the interface. The slope of the interface is always in the opposite direction to that of the groundwater surface. The interfacial slope is always the steeper, since the ratio of the two slopes is equal to the ratio of the density of the fresh water to the difference in density between the salt and fresh water. If the relative density of the salt water is assumed to have the nominal value of 1.025 with respect to the fresh water, the downward slope of the interface will be forty times the groundwater slope As stated previously, these considerations apply only to equilibrium conditions, and hence represent only the maximum salt water intrusion that would occur for a given set of continuing groundwater elevations.

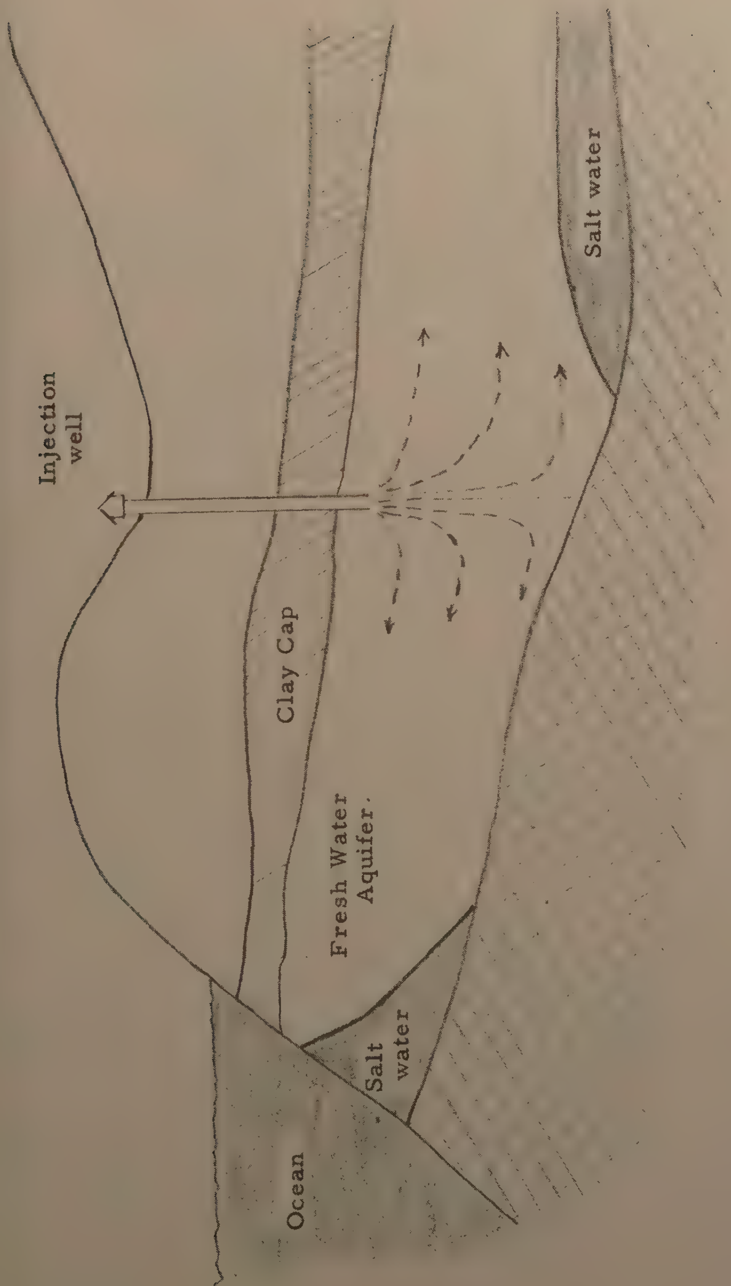

Fig. 1. Alternate Estimate of Ultimate Condition

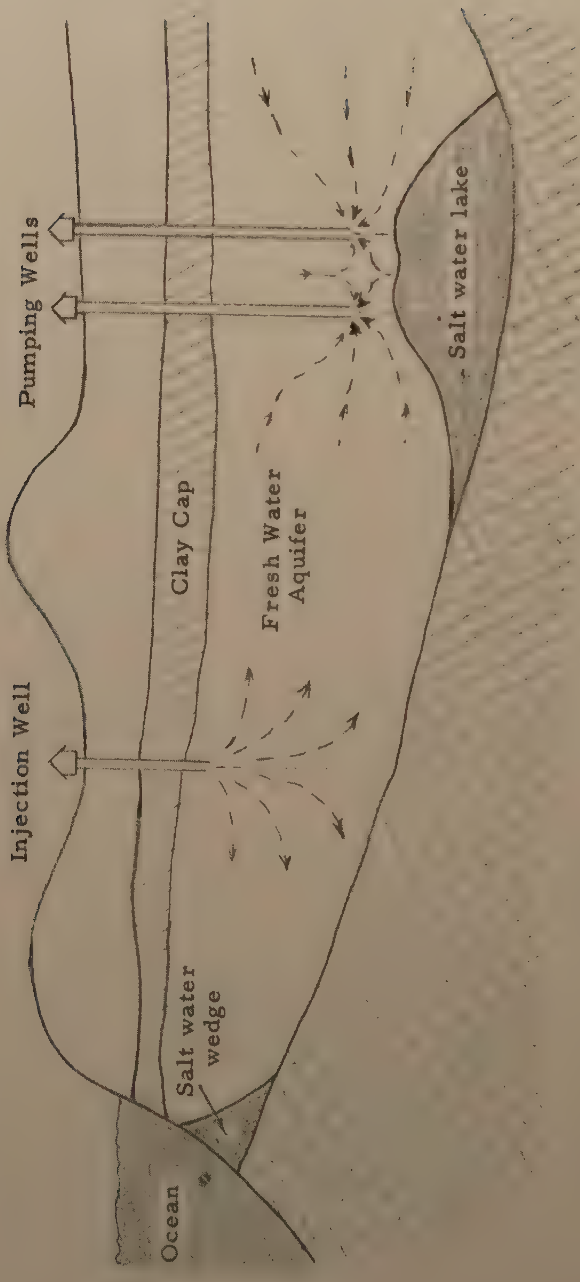

Fig. 2. Effect of Pumping on Interface of Salt Water Lake

Discussion of
"EXTENDING STREAMFLOW DATA"

by W. B. Langbein and C. H. Hardison
(Proc. Paper 826)

R. D. GOODRICH,[1] M. ASCE.—The paper on "Extending Streamflow Data" s one that the writer finds not only interesting but very timely and useful in onnection with the engineering investigations of the Upper Colorado River Commission. A great deal of its work is covered exactly by the title of this aper. The record of the authors wide experience in this field and their many uggestions for the proper application of mathematical and graphical correla-ion procedures should be especially useful wherever short term records nust necessarily be utilized extensively as in the water supply studies for is Commission. Not the least valuable feature of this paper is the authors iscussion of the limitations which are inherent in any method which can, at est, give only approximate results.

One very important problem which has arisen in this Commission's studies s that of fitting the short term records at newly established stations at im-ortant points on a stream into the water supply picture at the earliest prac-cal date, and, at the same time, to know how reliable the estimated quantities ill be when the records at the new station are extended back for perhaps 20 ears.

On the Dolores River in Colorado, it has been found desirable to discon-nue the gaging station at Gateway because of changes in the stream channel eriously affecting both gage and control. This station is a few miles east of e Colorado-Utah State line, and a new station has been established down-tream at about an equal distance west of the state boundary. After numerous ials and considerable study, it was found that when concurrent monthly re-ords in 1000 acre-foot units for three water years were plotted on log paper, the annual discharges were plotted with 10,000 acre-foot units, the points ll along the same general trend lines as the monthly records. Furthermore, robably because of the fact that the two stations are on the same stream only 5 miles apart, the trend of the plotted points was nearly along a 45° line. It, erefore, appeared possible to compute an equation for the regression curve hich would express the relation of monthly discharges at these stations with satisfactory degree of reliability. Could not the same identical equation be sed to give annual values of discharge simply by taking ten times the monthly it for the unit of annual flow?

To test this possibility, the discharge records at two locations on the olorado River about 75 miles apart were treated in the same manner. The onthly records for the water years 1951-2, 1952-3 and 1953-4 at Glenwood prings on the Colorado River were added to those on Roaring Fork for the tals at the upper station and the concurrent records at the station near ameo on the Colorado River were used for the record station for which

[1] Eng. Consultant, Upper Colorado River Comm., Grand Junction, Colo.

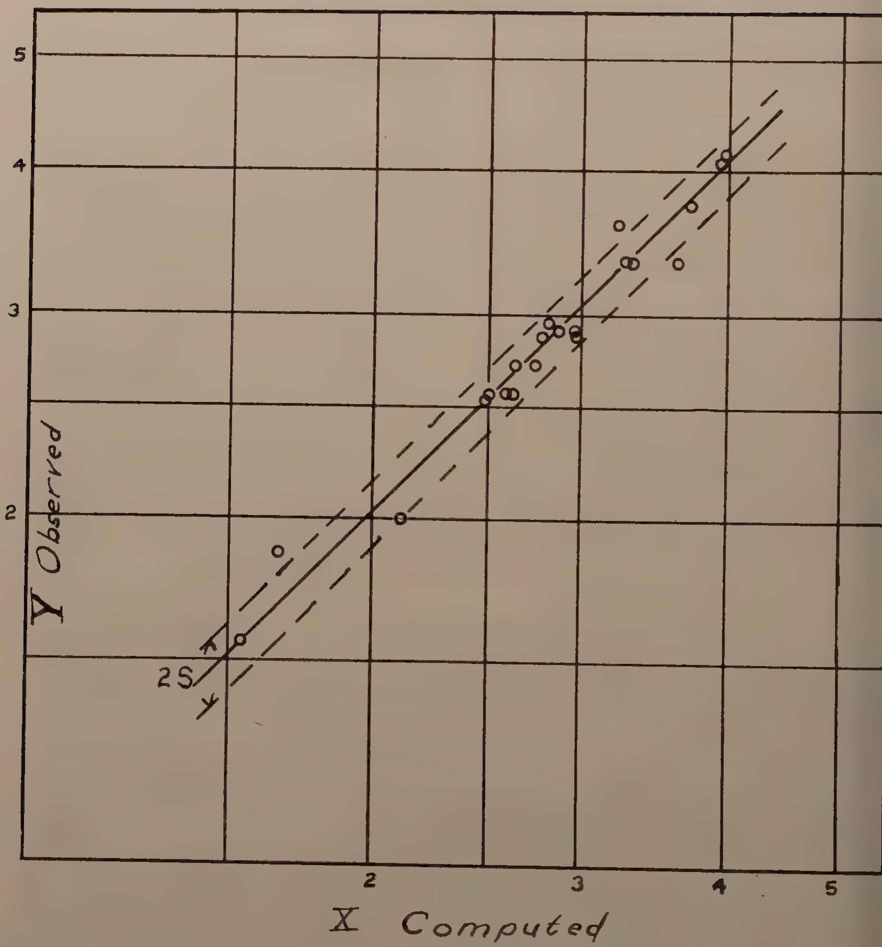

Fig. 1. Comparison of Computed and Observed Discharges

estimates of annual discharge were to be carried back for a total period of 21 years, that is to 1934, when the Cameo station was started. This gave a series of 39 items including the three annual discharges. These records include one year of high flow, one of very low flow and one of nearly average discharge. The standard error of estimate for this series, using discharges and not their logarithms, was 25.1 units of flow or 10.8%. In order to have a series that was as homogenous as possible, three of the monthly records were omitted for the next computation. One of these records had a departure of one standard error, one a departure of two and the third a departure of more than three standard errors. This first trial indicated beyond question that the use of the logarithms of the discharge would give a distribution that, as the authors state, would be more nearly normal than if the discharges themselves are used. The following equations were derived. Logarithms used are to base 10.

$$\log Y = 0.95989 \log X + 0.13765$$
$$Y = 1.373\, X^{0.960}$$

Based on the 33 monthly records and the 3 annual ones, the adjusted coefficient of correlation was $\bar{r} = 0.9976$ and the standard error was $\bar{S} = \pm 0.03161$ log units or + 7.55% and - 7.00%. When the actual annual estimates were computed and compared with the 21 years records available at the Cameo station, the results turned out better than indicated above. The mean absolute error was only 4.8% indicating a standard error for this period of ± 6%. In 21 years, 6 or 7 times one might expect differences of 6% or more. However in 2 years, 1934, a low year, and in 1942, a high year, the error near 6% (-5.7%), and five times the difference between recorded and computed flows was less than 1%. More trials and study will be necessary before one may safely adopt this use of a single correlation equation for both monthly and annual discharges, but under similar conditions to those described the procedure outlined seems to have sufficient promise to justify much further study.

The writer has been very much interested in the study of this paper and has found much of value which can be used in his own work and little to criticize. Personally, whenever possible we prefer to use mathematical processes with graphical methods for illustration and checks. The accompanying diagram gives a comparison of the records with the computed discharges obtained in the above test. The authors are to be highly commended. The writers own less technical studies tend to confirm their conclusions.

CARROLL F. MERRIAM[1] and E. T. SCHULEEN[2] M. ASCE.—Those who have had extensive experience in studying the hydrology of the Susquehanna River will approve the dominant thought expressed by the authors: that by means of extending short term records on the basis of correlation to long term base records, the knowledge of streamflow can be advanced in the most economical manner. The proposed use of roving satellite stations is the sensible solution to the critical demand for more and more basic hydrologic data. The engineering profession should encourage the U. S. Geological Survey in furthering such a program.

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1. Prospect Harbor, Maine, formerly with Pennsylvania Water & Power Co.
2. Assistant to Manager, Safe Harbor Water & Power Corp., Conestoga, Pa.

For a number of years the Pennsylvania Water & Power Company[3] and the Safe Harbor Water Power Corporation had been developing methods for analysis of past records of streamflow at gaging stations and hydroelectric power stations in the Susquehanna basin. It is important that one of these procedures is very similar to what the authors call "equal per cent duration," and which they point out gives results, as far as can be shown in Fig. 2, identical with the correlation method.

The principal differences in practice were in the use of daily instead of monthly data and the use of stage instead of the logarithm of discharge. The obvious objection to daily data, the tremendous amount of labor involved, was overcome by business machine methods. Punch cards, for which many other statistical applications have been found, were used to record the daily discharges at 21 stations for the past 24 years. Periods of time were selected for which records at the stations to be compared were continuous and unaffected by ice or other abnormalities. The sorting machine listed the discharges in parallel columns and in ascending order of magnitude. Inasmuch as there were the same number of entries in each column, those appearing on the same line had equal frequency of occurrence. In this phase discharge was used for the reason that the given data were so expressed, but conversion was made to stage on the lists by reference to the rating curve in effect at the time of observation. Plotting the stages shown on each line, one against the other, resulted in a relationship curve of equal probability of occurrence, similar to that shown at the extreme left of Fig. 2. This relationship could then be used to estimate stages and discharges at the station with a short term record by reference to those at the station with the longer record. Actually it is used to estimate the stage that is most likely to occur at a down stream station on the basis of an observed stage at an upstream station.

Stage was used, not for the sake of normalizing the decided skew in the discharge frequency, which the authors accomplished through use of a log scale of discharge, but rather because the work was of such a nature that shifts that had been made in rating curves from time to time would have caused substantial variations in the results.

The authors seem somewhat surprised that the several methods give almost identical results. Their conclusions appear to indicate that what the writers, in the development of stage relation curves, had termed "conjugate stages" or "conjugate discharges," and which might be defined as stages or discharges having the same frequency of occurrence, do have greater reality than originally supposed. This concept should prove very useful in solving flood routing problems.

If one were interested only in long term mean discharge, a much easier procedure than that proposed by the authors is the use of the double mass curve. The accumulated monthly inches of runoff for the station being investigated are plotted against the accumulated runoff at a base station, or better yet, against the mass curve of the basin runoff pattern, which, in the case of the Susquehanna, consisted of the average of approximately 100 gages weighted by the square root of the drainage area. The slope of the double mass curve gives the relation of the mean runoff at the station to that of the basin. This method has been extensively used in studies of the Susquehanna to derive useful estimates from many records altogether too short or fragmentary to be depended upon otherwise. In this way useful results have been

3. Now merged into Pennsylvania Power & Light Company.

salvaged from about 50 gages since discontinued, many of them with only a few years of record. The reasonableness of these estimates has been verified by comparing runoffs from gaged areas with the residual runoff from ungaged areas on a "family tree" diagram.[4] The success of the methods lead the writers to conclude that the policy of roving satellite stations is sound and certainly should be adopted.

It should be emphasized that this is not an end in itself, but rather the beginning of a long road toward greater achievements in the field of applied hydrology. Past experience has also shown that while the U. S. Geological Survey should be encouraged to forge ahead, private industry has much to gain by placing a shoulder to the wheel.

WILLARD M. SNYDER,* A.M. ASCE.—A good, objective treatment of an example of the use of statistics in hydrology is presented. Certainly carefully selected drainage areas should be capable of sufficiently good correlation so that base stations could serve as "predictors" for satellite sites with rather closely controlled variances of prediction. The similarity in characteristics of adjacent streams has been used for such varied purposes as interpolation for missing records, derivation of synthetic unit hydrographs, and preparation of regional maps of streamflow parameters.

The authors invite discussion on particular points of their methodology when they state "------the details of the techniques are far from completely developed." There can be no serious question of the statement that statistics should be the tool of hydrology, rather than vice versa. This idea should be extended to state that all phases of methematical and numerical methods are but tools. The basic science must provide the physical logic.

The graphical statistical correlations are particularly useful when the problem to be analyzed combines small samples with very evident correlation of a few variables. But when samples become large, and when causation is a combination of many separate parameters and iterative solutions required for graphic multiple correlation become agonizingly tedious. Under the latter conditions more economical analyses can probably be accomplished using a product-accumulating desk calculator and following the conventional least-squares techniques for multiple correlation. It is acknowledged at this point, that techniques for selection of proper mathematical models for hydrologic analyses are also far from completely developed.

It appears necessary to distinguish between statistical tests based on frequency distributions of uncorrelated data, and tests based on the residual errors in data after adjustment by regression analysis. Highly skewed frequency distributions must be "normalized" for testing, and a logarithmic transformation apparently accomplishes this purpose. But it is not self evident that the frequency distributions must be normalized for regression analysis. It is more important** that the residual errors be normally distributed with equal variance along the line of regression. Least-squares fitting by

4. Progress Report On The Analysis of Rainfall Data, Carroll F. Merriam, Transactions of the American Geophysical Union, 1938, p. 532.
* Civ. Engr. (Hydr.) Tennessee Valley Authority, Hydraulic Data Branch, Hydrology Section, Knoxville, Tenn.
** Anderson, R. L., and Bancroft, T. A. Statistical Theory in Research. McGraw-Hill, 1952. p. 153.

definition produces minimum residual variance, no matter what the distribution of the variates, and approximate statistical tests may be based on these results. The seriousness of the approximation is difficult to determine and in critical instances should be referred to mathematical statisticians.

The authors use the expression "standard error of estimate" for the square root of the variance of residuals. This nomenclature is possibly misleading. A better expression is "standard deviation of residuals." It is, of course, inherent in the theory of least-squares that the residual variance be uniformly distributed along the line of regression. When fitting is done by logarithms, the fitting does produce a standard deviation of residuals which is a constant percentage of the dependent variable after transformation back to the original units.

It does not seem correct to pass over the assumption that "the mean line is correctly located." The analyst cannot know whether or not this is true. Consequently, he cannot know whether approximately two-thirds of future occurrences will be within a uniform band of plus and minus one standard deviation from the fitted line. When the regression is used for prediction of future events, the variance of the prediction is the residual variance plus the variance of the fitted line.

The variance for a single predicted value of Y' for a particular value X' is

$$\left[1 + \frac{1}{n} + \frac{(X' - \bar{X})^2}{S(X - \bar{X})^2}\right] s^2$$

and the variance for the average of all predicted values of Y' for the particular value X' is

$$\left[\frac{1}{n} + \frac{(X' - \bar{X})^2}{S(X - \bar{X})^2}\right] s^2$$

where s^2 is the variance of the residuals

$\bar{X}$ is the mean of X

and $S(X - \bar{X})^2$ is the sum of squares about $\bar{X}$.

From these formulas it can be seen that the variance of prediction depends upon the particular value of X' on which the prediction is based. Even for $X' = \bar{X}$ (prediction at the mean value) the variance of individual predictions is not the same as the variance of residuals, but depends upon the size of the sample. The variance for the average of all predictions does become s^2/n for $X' = \bar{X}$. The "engineering" preference for errors as percentages is thus not justified for variances of prediction but only for variances of residuals of fitting. The authors point out the break in variance between original record and predictions.

The distinction between the variance residual to fitting and the variance of prediction should be maintained for the "unregressed" model.

The use of the geometric mean to obtain a unique line of relation between

two variables has been mentioned by other authors.[1] The slope of the fitted line by this method is the geometric mean of the two slopes obtained by minimizing the residual sum of squares along first the vertical and then the horizontal axes. Minimizing the product of the vertical and horizontal residuals produces the same result. It is not clear that the graphical method of obtaining the medians of vertical and horizontal strips would result in the same line. Any conclusions based on analogy between the two methods would have to be considered approximations if the lines are different, since the distribution of residual errors, and hence residual variance, will vary with the fitted line.

1. Gumbel, E. J. "Statistical Theory of Extreme Values and Some Practical Applications." National Bureau of Standards Applied Mathematics Series No. 33 p. 16.

Discussion of "LUNAR-CYCLE MEASUREMENT OF ESTUARINE FLOWS"

by Irvin M. Ingerson
(Proc. Paper 836)

FRANKLIN C. CRAIG,[1] A.M. ASCE.—This paper is an important contribution to a subject that presents many complex problems. Much of the planning for the future development of the Central Valley area depends upon a knowledge of how much flow must be maintained in the lower Sacramento River channel. The measurement of flow discussed by Mr. Ingerson is therefore of considerable economic importance.

Of prime interest is the high degree of accuracy that must be attained if results are to be truly indicative of net outflow. In a tidal cycle measurement accidental errors tend to become negligible because of the large number of observations that are made, assuming of course that exactly the same points of observation are maintained and other precautions are taken to assure accurate work. Systematic errors which may apply unequally to flood and ebb tide flows offer the greatest possibility of producing erroneous results. The assumed figures of Table 1, which are commensurate to actual volumes of flow, illustrate the importance of avoiding errors of this type. The table is based throughout on hypothetical flood flows of 200,000 cfs and ebb flows of 205,000 cfs.

Craig on Ingerson, Table 1 -- Comparative Error Possibilities

Measured Average Flood Flow cfs	Assumed Percent in Error	Measured Average Ebb Flow cfs	Assumed Percent in Error	Indicated Net Outflow cfs	Error in Net Outflow cfs
200,000	0	205,000	0	5,000	0
204,000	+2	209,100	+2	5,100	100
220,000	+10	225,500	+10	5,500	500
204,000	+2	205,000	0	1,000	4,000
204,000	+2	200,900	-2	-3,100	8,100
196,000	-2	209,100	+2	13,100	8,100

Fairly large errors can be tolerated if they apply equally to flood and ebb flows but even very small errors can be disastrous if they are of varying magnitude and especially of opposing sign.

Possible sources of systematic errors which might not apply equally to flood and ebb flows are (1) variable density effects caused by incomplete mixing of salt and fresh water, and (2) wind effects.

The average of point velocities at .2 and .8 depth does not always represent the true mean velocity in the vertical. This has been found to be especially true in some tidal streams where the heavier salt water tends to move

1. Hyd. Engr., Geological Survey, U. S. Dept. of the Interior, Sacramento, Calif.

along the bottom quite independently of the lighter fresh water on top. Flood and ebb tide flows are sometimes affected differently by variations in density. A limited number of velocity observations were made at each tenth of the depth, during the course of the measurement that has been described by Mr. Ingerson. These special observations suggest that the mean of the .2 and .8 depth observations may be adequate at the measuring section that was used, at least under the conditions of tide, flow, and wind, at which the comparisons were made. Salt and fresh water mixing action is apparently more complete here than it is in some tidal streams.

Some lateral shifting or wandering of currents in a direction not perpendicular to the measuring section occurs during periods of low velocities. Such errors as might be introduced from this source would have a tendency to be compensating during periods of flood and ebb flow.

Little is known concerning the magnitude of error that may be introduced by wind. Abnormal movement of meter cups induced by vertical boat movement, distortion of vertical velocity relationships, shifting currents, and change in the normal mixing action of fresh and salt water are all possible effects of wind. Wind movement often varies; hence, errors of unequal magnitude may occur during flood and ebb flow periods. In the absence of definite knowledge concerning wind effects, it would seem to be good practice to place the most confidence in observations made during periods of minimum wind movement. If winds occur during a measurement, it would be good practice to extend the period of measurement well into a period of calm because of the effects of wind on channel storage.

One of the purposes of the measurement of flow that has been described in the paper is to determine the amount of flow which may accrete to the stream from unmeasurable ground-water sources. The amount of these accretions may vary seasonally, or from year to year because of change in ground water conditions or consumptive use by crops. For this reason a continuous record of flow is desirable, and consideration should be given to possible methods of rating the channel.

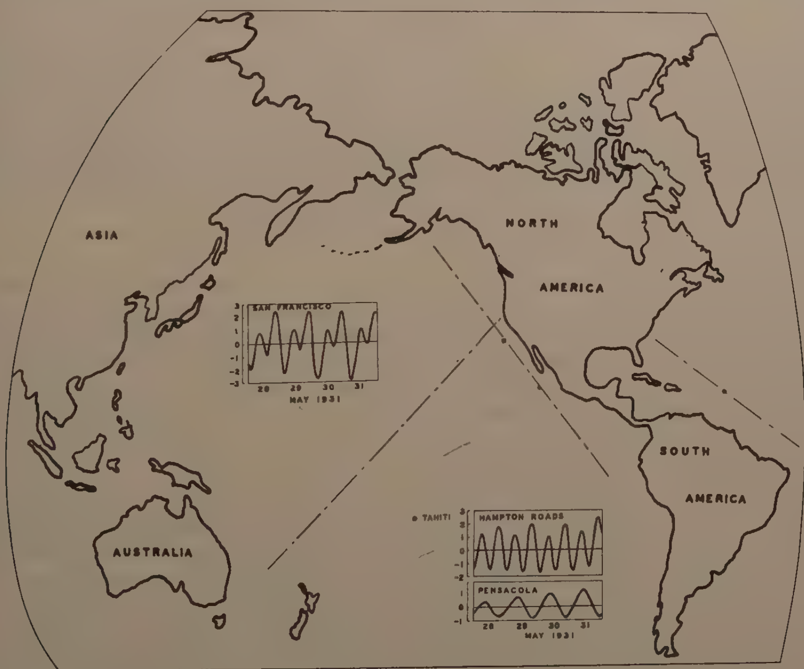
First consideration might be given to a proposal that the salinity gradient between two ends of a selected reach may be used as a factor in determining outflow. If the salinity gradient could be thus utilized, it would furnish the most economical method of obtaining continuous records of flow because it would eliminate the need for frequent expensive current meter measurements. Incomplete mixing of fresh and salt water would seem to present a major source of error in determining the salinity gradient. This would be especially true if the measurements of salinity bear a different relation to the true salinity during flood and ebb flow periods.

The nature of a tidal stream is somewhat different from that of an ordinary stream of mild slope entering a lake. The ordinary stream has a water surface profile that is concave upward within the limits of backwater effect and a definite relationship exists between stage, slope and flow. In a tidal stream, however, the profile of the mean water surface tends to arch upward by an amount which varies with the range of tide levels, the volume of flow, the level of the ocean, and other factors. This variable arching action enters into the problem of rating a tidal stream.

The modern theory of tidal action in the ocean is that it consists of the rocking up and down of water in large natural basins across virtually tideless nodes. Tidal action is said to develop when the natural period of oscillation within the basin coincides with that of the tide-producing forces; principally

the moon and the sun. Tidal action at San Francisco (writer's Fig. 1) is complex because the rocking action takes place along two axes.[1] A tidal pattern results that is a mixture of the diurnal and semi-diurnal types of tides. These two basic tidal patterns are illustrated by tide graphs for Pensacola, Florida and Hampton Roads, Virginia. The mixed tidal pattern is characterized by succeeding tides that frequently vary considerably in magnitude and that do not usually oscillate about mean sea level. These characteristics, of course, add to the complexities of streamflow measurement and rating problems.

When the ocean surface rocks upward, a wave of translation[2] is said to be induced within the estuary which is propagated upstream until its energy becomes depleted by friction and by the force of the downflowing current. Likewise when the ocean rocks downward, a negative wave of translation is induced which is also propagated upstream. However, the energy of the negative wave becomes depleted much more rapidly than that of the positive wave because



Craig on Ingerson, Fig. 1—Tidal Patterns

the negative wave travels in shallower water and is therefore subject to greater frictional effects. Also, because more water must be moved under the influence of the negative wave, it has more work to do. It is the unequal depletion of the energy of the positive and negative waves that causes the

1. Applied Hydrology, by Linsley, Kohler, and Paulhus; McGraw Hill Book Co., New York, 1949.
2. "Flow of Water in Tidal Canals" Col. Earl I. Brown, Transactions American Society of Civil Engineers, 1932.

mean daily water surface profile of the tidal stream to tend to arch upward, thus creating a head to achieve a balance of wave force.

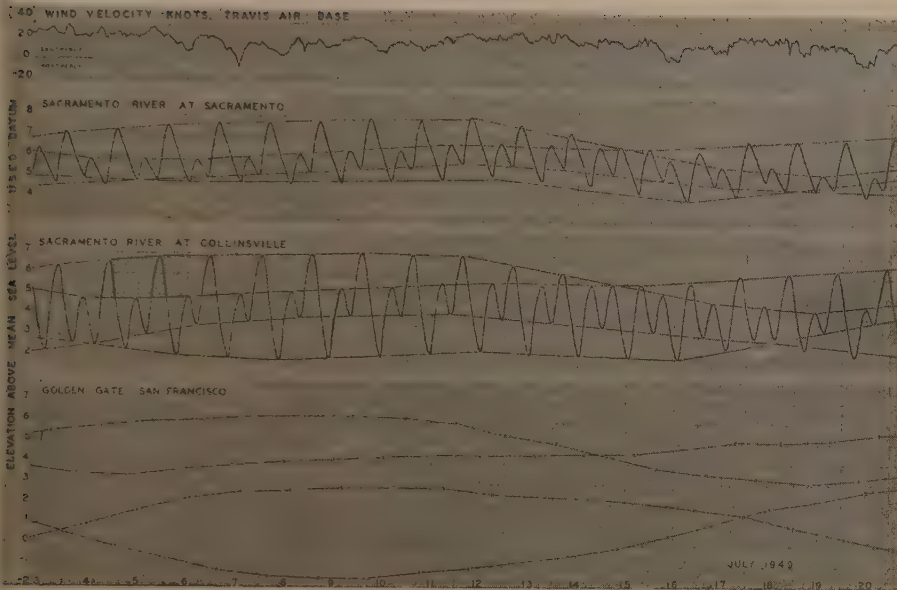
The unequal rate of depletion of the energy of positive and negative waves creates rather special rating problems as is evidenced by the operation of a gaging station on the Sacramento River at Sacramento. Here the stream is subject to a range in tidal action of about two feet but not to a reversal in the direction of flow. A reach of channel extending twenty miles downstream from Sacramento to Snodgrass Slough has been used for rating purposes. In this reach the net effect of tidal action over a period of a complete tidal cycle is much the same as that which would be created from any source producing variable backwater effects, except that the forces which create abnormal stages are active within the rating reach and may create even more abnormality in stage at the upstream end of the rating reach than they do at the downstream end of the reach. This is in marked contrast to the behavior of ordinary backwater effects. The result is that the fall through the reach is not a dependable measure of the effective slope at any point in the reach. This places a definite limitation on the type of rating relationship which may be used.

Considerable success has been had in rating the Sacramento River at Sacramento by a rating fall method, a standard method used in rating many backwater-affected streams. This method makes use of a ratio between measured fall and rating fall, and a ratio between actual discharge and a rating discharge, in accounting for the effect of variable abnormalities in stage introduced by tidal action. Experience in the use of the method under a wide range of conditions is needed before its dependability for use on tide affected streams can be definitely established.

Considerable success has also been had in rating the Sacramento River at Sacramento by methods which develop corrections either to abnormal fall or to abnormal stage. These methods have as their basis the depletion of wave energy as measured by the depletion in range of stage (between crests of positive and succeeding negative waves), from the downstream to the upstream end of the rating reach. The corrections to fall are applied to a unit-fall rating extended downward as a straight line and the corrections to stage are applied to a base rating curve.

The problem of rating the lower Sacramento River channel is complex because of the physical characteristics of the channel and because the flow is in two directions. There are no reaches in the lower Sacramento River channel that are well adapted to the methods that have been used to rate the Sacramento River at Sacramento. With the exception of a few short, somewhat constricted reaches (author's Fig. 1), the lower channel is very wide and is in the nature of a bay rather than a river. Since little fall develops during a portion of the tidal cycle, the minimum length reach that could be used if fall were a factor in the relationship, would be about fifteen miles unless special stage recording equipment were developed. In a reach of this length, a large portion of the flow would be continually entering or leaving channel storage, hence it is evident that special methods, which consider changes in channel storage, would be required to rate this channel.

Also of concern is the effect of wind. Strong upstream winds prevail during much of the time, but light or strong, upstream or downstream winds may occur within a matter of hours. The fetch is about 30 miles and waves several feet high often develop. Of more concern than surface waves are small changes in mean stage and rates and distribution of flow which are caused by



Craig on Ingerson, Fig. 2—Changes in Tide Pattern due to wind effects and normal depletion.

wind. Here again the danger is that flood and ebb flow relationships will be affected differently and especially by winds of varying magnitude. The importance of very small changes in normal relationships is evident when it is considered that the entire range of flow, from zero to about 400,000 cfs takes place within a range of stage of only about 5 feet. A net outflow of 5,000 cfs would be represented by a change in stage of only about a tenth of a foot.

Some idea of the magnitude of change in stage that is the result of wind can be gained from a study of the change in pattern of tidal action as it moves upstream. Shown in the writer's Fig. 2 are recorded tide crests at San Francisco and gage-height graphs at Collinsville and Sacramento. Lines have been drawn connecting wave crests in order to emphasize the pattern. At San Francisco the line connecting the crests of the negative waves makes a broad downward sweeping arc. At Collinsville this line reaches an unnatural looking peak on July 12 which appears to be associated with wind action. The most pronounced effects can be noticed when the wind quickly subsides, as on July 20. It appears that wind may affect normal river levels by as much as several tenths of a foot. Since wind effects depend on both duration and magnitude, they are difficult to evaluate. Unless the rating method properly considers the effects of wind, it would be of questionable value except under mild wind conditions.

Since changes in upstream channel storage are continually in progress, flow in the lower Sacramento River must be computed through at least a fortnightly and preferably a lunar monthly period in order to arrive at a dependable value of net outflow.

Quite probably, dependable records of daily flow might be obtained by the use of a number of continuously operated current meters suspended from pile

structures spaced at intervals across the stream. The velocities at the points at which the meters were suspended would be related to the average velocity of the stream as predetermined from regular measurements. Whether a few or many current meters and recorders would be required might be determined from an analysis of the series of measurements made in September 1954 and described in the paper. If a large number of meters were required, the system, of course, would be very costly. It should, however, produce records that would be dependable except during periods of very heavy winds.

The limitations of the stationary meter in measuring extremely low velocities and the difficulty in determining direction of flow are recognized. A well balanced Price type meter in excellent mechanical conditon is dependable in measurement of velocities of 0.3 foot per second. The moving boat method as described in the paper has shown that, on an average, the pattern of velocities in a tidal cycle, from a velocity of 0.3 foot per second through the period of slack-water to a velocity of 0.3 foot per second in the opposite direction, is very nearly a straight line. If the current meter were suspended from a rigid structure, there should be no appreciable error in adjusting observed velocities to this pattern during the relatively short periods when very low velocities prevail. This procedure would also eliminate need for concern regarding direction of flow. The use of a fixed meter has its place in the measurement of tidal flows if its limitations are borne in mind.

The instrumentation research engineers are making rapid strides toward perfecting new techniques and equipment for measuring extremely low water velocities. One device being tested by the Geological Survey seems to offer great promise for measuring point-velocity by the rate of heat loss from a probe to the water flowing past it. This device is hoped to be free of some of the difficulties experienced with earlier versions of the hot-wire anemometer. Other developments of ultrasonic wave velocity measuring devices[1] may, in the future, permit recording the instantaneous mean water velocity on a line (sound wave path) across rivers. Such new devices may be of material assistance some day soon in measuring complex tide-affected streams like the Sacramento River.

The moving boat method and the special methods of analysis of data that have been presented in Mr. Ingerson's paper are ingenious solutions to problems not often encountered in ordinary stream gaging operations. These methods may prove to be of value in other special situations such as the measurement of streams of very low velocities and especially in the measurement of flow of other tidal streams.

In view of the new data obtained in the measurement described in the paper, it is believed that additional observations and studies in line with the avenues of approach presented in the foregoing discussion might point the way to the development of a reliable rating of the net outflow from the Sacramento-San Joaquin Delta.

L. J. TISON.[2]—A first very interesting method presented in this paper is the new "moving boat method" of using a standard current-meter to measure the low velocities.

A second remarkable part is the description of the "Delta Outflow

1. Swengel, R. C., Hess, W. B., Waldorf, S. K., "The Ultrasonic Measurement of Hydraulic Turbine Discharge"; A.S.M.E. paper No. 54 A 54.
2. Prof., Univ. of Ghent, Belgium.

Measurement" during a lunar-cycle, between times of equal tidal-volume. By choosing-times of equal tidal-volume, said the author, the item of tidal-volume goes out of the hydrologic equation and this equation is reduced to the terms of accurately measured flows and to only one unknown: the net use of water in the Delta area.

A theory presented in 1908 by a Belgian engineer Van Brabandt, suggests another method for the determination of the net use of water in the Delta.

Theorem of Van Brabandt

The volume between two surfaces, the first one joining the heights of the slack-water reversal on the ebb in the different cross sections upstream the cross section where the flood flow is to be established, the second one joining the heights of the following slack-water reversal on the flow, is equal the flood flow modified by the volume of the lateral flow.

The demonstration presented by Van Brabandt was a very long one and was based on geometrical considerations. We present the following demonstration.

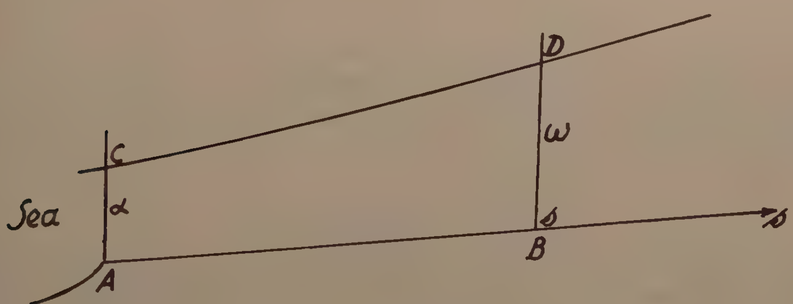


Fig. 1

AB is the bottom of the river and CD is a simultaneous tidal line between two cross sections α en ω : α is the fixed last cross section of the river, while the cross section ω varies with

The tidal-volume change $-\overset{t+\Delta t}{\underset{t}{V}}$ equal the algebraic summation of the volume flowing laterally $\overset{t+\Delta t}{\underset{t}{\lambda}}$ and of the volume flowing through the cross sections α : $\overset{t+\Delta t}{\underset{t}{A}}$ and ω : $\overset{t+\Delta t}{\underset{t}{\Omega}}$:

$$\overset{t+\Delta t}{\underset{t}{V}} + \overset{t+\Delta t}{\underset{t}{A}} + \overset{t+\Delta t}{\underset{t}{\Omega}} + \overset{t+\Delta t}{\underset{t}{\lambda}} = 0 \qquad (1)$$

hence:

$$\frac{\partial^2(V)}{\partial t \partial s} + \frac{\partial^2(A)}{\partial t \partial s} + \frac{\partial^2 \Omega}{\partial t \partial s} + \frac{\partial^2 \lambda}{\partial t \partial s} = 0 \tag{2}$$

The volume flowing through the fixed cross section α is not altered by the variation of s and $\frac{\partial A}{\partial s}$ equal zero.
Hence equation (2) reduces to:

$$\frac{\partial^2 V}{\partial t \partial s} + \frac{\partial^2 \Omega}{\partial t \partial s} + \frac{\partial^2 \lambda}{\partial t \partial s} = 0 \tag{3}$$

On the other side the flood flow to be determined in an arbitrary cross section s is:

$$F_f = \int_{t_j}^{t_f} q_s \, dt \tag{4}$$

t_f is the time of the slack-water reversal on the ebb and t_j is the time of the following slack-water reversal on the flood, in the cross-section ω ; q_s is the instantaneous flow in the same cross-section.

We obtain, taking the derivative of (4):

$$\frac{\partial F_f}{\partial s} = \frac{\partial}{\partial s} \int_{t_j}^{t_f} q_s \, dt = \int_{t_j}^{t_f} \frac{\partial q_s}{\partial t} \, dt + (q_s)_{t_f} - (q_s)_{t_j} \tag{5}$$

The two last terms are the values of the instantaneous flows in the cross section ω for the times of the slack-water reversal on the flow and of the slack-water reversal on the ebb respectively. These values therefore equal zero and we write:

$$\frac{\partial F_f}{\partial s} = \frac{\partial}{\partial s} \int_{t_j}^{t_f} q_s \, dt = \int_{t_j}^{t_f} \frac{\partial q_s}{\partial s} \, dt = \int_{t_j}^{t_f} \frac{\partial^2(\Omega)}{\partial s \partial t} \, dt \tag{6}$$

while

$$q_s = \frac{\partial(\Omega)}{\partial t} \tag{7}$$

Owing to the equation (3), we may write:

$$\frac{\partial F_f}{\partial s} = -\int_{t_j}^{t_f} \frac{\partial^2(V)}{\partial s\partial t} - \int_{t_j}^{t_f} \frac{\partial^2(\lambda)}{\partial s\partial t}\, dt \qquad (8)$$

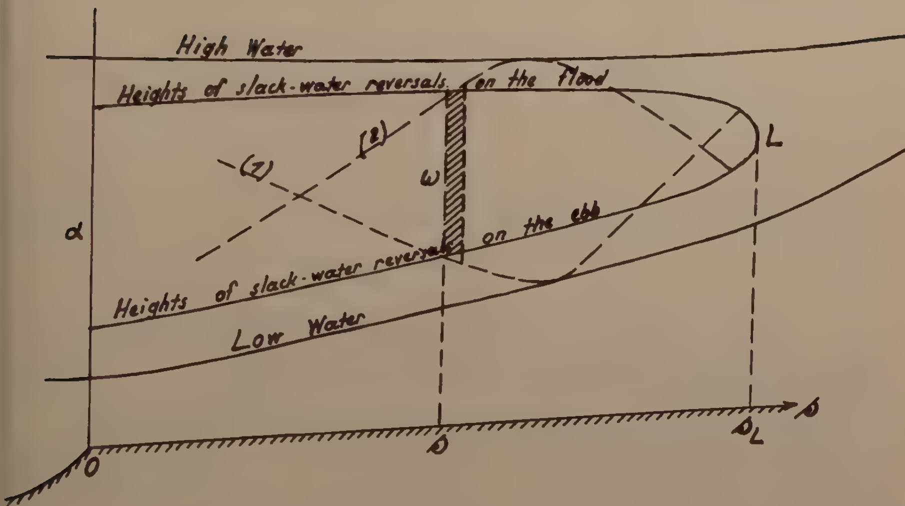


Fig. 2

Fig. 2 shows different characteristic curves for a tidal river. The cross section L is the first cross section presenting slack-water reversals. Upstream this cross section, there is no more flood flow.

Integrating the equation (8) between the cross-sections α en L , we obtain:

$$-(F_f)_\alpha = \int_0^{s_L} ds \int_{t_j}^{t_f} \frac{\partial^2(V)}{\partial s\partial t}\, dt - \int_0^{s_L} ds \int_{t_j}^{t_f} \frac{\partial^2(\lambda)}{\partial s\partial t}\, dt \qquad (9)$$

the flood flow indeed equal zero in the cross section L.

$$ds \int_{t_j}^{t_f} \frac{\partial^2(V)}{\partial s\partial t}$$ is the tidal-volume change, between the cross sections

s en s + ds, from the time of the slack-water reversal on the ebb in the section s and the time of the slack-water reversal on the flood in the same section.

It is therefore represented by the hatched part of the fig. 3 where the

curves (1) and (2) are the simultaneous tidal lines for t_j and t_f respectively.

$\int_0^{s_L} ds \int_{t_j}^{t_f} \frac{\partial^2 (V)}{\partial s \partial t} dt$ is consequently the volume between two surfaces, the first one joining the heights of the slack-water reversals on the ebb, the second one joining the heights of the slack-water reversals on the flood.

It is easy to see that $\int_0^{s_L} ds \int_{t_j}^{t_f} \frac{\partial^2 (\lambda)}{\partial s \partial t} dt$ is the lateral flow. In the problem of the Delta at the confluence of the Sacramento River and the San Joaquin River, this lateral flow is the part of the net use in the Delta area during a flood period and between the sea and the first cross-section L presenting slack-water reversals.

This part of the net use in the Delta is therefore the difference between flood flow and the volume of Van Brabandt.

The result of this method cannot be compared with the result of the author: this last result is an average for a total tidal-cycle and for the whole Delta.

However we feel that our method presents some interest.

Discussion of
"INTEGRATING THE EQUATION OF GRADUALLY VARIED FLOW"

by Ven Te Chow
(Proc. Paper 838)

CLINT J. KEIFER,[1] A.M. ASCE and HENRY HSIEN CHU,[2] A.M. ASCE.—Integrating the equation of gradually varied flow has been attempted by many investigators and success has been limited only to certain approximations or for certain specific type of channels. The author is to be complimented for advancing the methodology in the fact that only one set of tables need to be used to solve both the friction loss and velocity head change terms. However, the writers experience some disappointment in trying the author's method for the closed conduit case. When the discharge through a closed conduit is greater by a few percent than $K\sqrt{S_0}$ for the full section, which should not be considered as an infrequent occurrence, the normal depth y_n becomes non-existent and rendering the final formula, Equation (38) meaningless. Even within the realm of its applicability, the writers attempted to test the steadiness of exponents N and J which should remain reasonably unchanged for any considered range of proportional depth y/D. Figure 6 shows a plot of N, M, and J versus y/D for the circular section. (The two broken curves, m and N/m will be explained later). The N and M curve was already presented by the author in Figures 3 and 4, but for a smaller range. The writers are rather surprised at the rapid variation of the exponent J for any y/D beyond 0.4. The J value approaches positive or negative infinity as the proportional depth nears 0.65 from below or above respectively.

Lee and others(17) presented a very similar method in which they arrived at the following equation, using the author's notations

$$x = -\frac{y_n}{S_o}\left[u - F(u, N) + \frac{V_n^2}{2gy_n} F\left(u^{-m}, \frac{N}{m}\right)\right] + \text{CONSTANT} \qquad (41)$$

where m is another hydraulic exponent defined by

$$A^2 \propto y^m \qquad (42)$$

which is similar to Equation (37) except in the third term, the varied flow function, N/m has been chosen as the exponent instead of J. It can be shown that

$$m = \frac{2Ty}{A} \qquad (43)$$

1. Senior Sewer Designing Engr., Bureau of Eng., Dept. of Public Works, Chicago, Ill.
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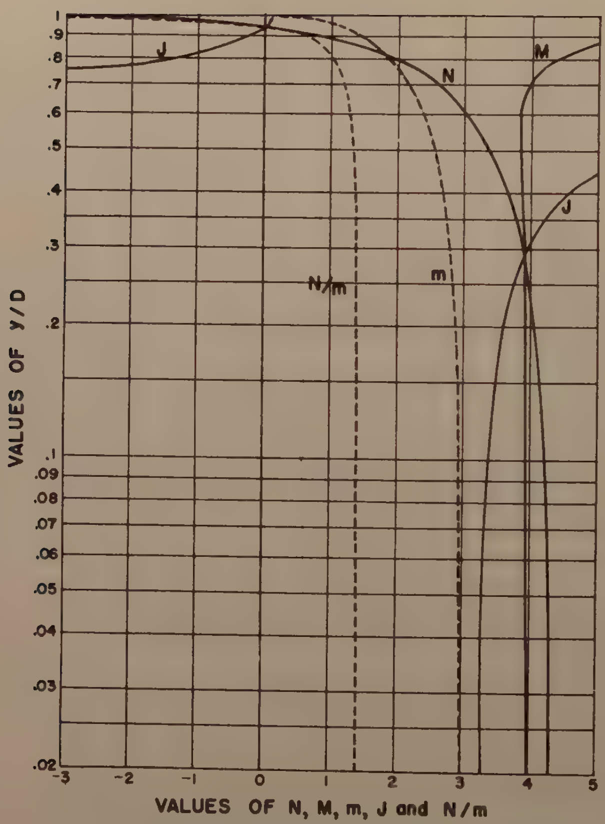

Fig. 6. CURVES OF HYDRAULIC EXPONENTS FOR CIRCULAR SECTIONS

A plot of m and N/m versus y/D are also shown in Figure 6 as dotted curves. It seems to the writers the N/m curve possesses better steadiness than the J curve and Lee's method should be good for y/D up to 0.8. However, it still occurs to the writers that the closed conduits actually present a different problem which has not been solved satisfactorily for the region near crown by any method presuming constant hydraulic exponents. The writers presented a different approach(19) devised for closed conduit primarily, but the principles hold true for open channels also.

As far as open channel cases are considered, the writers wish to see a graph showing the variation of J with respect to y/b using z as parameter. That will provide a better guide to those in the profession who intend to adopt the author's method and want to know where they stand on the matter of accuracy.

REFERENCES

17. "Gradually Varied Flow in Uniform Channel on Mild Slope" Ming Lee, Harold E. Babbitt and E. Robert Baumann, University of Illinois Engineering Experiment Station Bulletin Series No. 404, Vol. 50, No. 28, Nov. 1952.

19. "Backwater Function by Numerical Integration" Clint J. Keifer and Henry Hsien Chu, American Society of Civil Engineers, Transactions 1955.

CORRECTIONS

P. 2 - Equation (3) Denominator at right side "$1+\frac{d}{dx}\left(\frac{V^2}{2g}\right)$" should be "$1+\frac{d}{dy}\left(\frac{V^2}{2g}\right)$"

P. 3 - Equation (13) Right side "$\frac{Z_c^2}{Z^2}$" should be "$-\frac{Z_c^2}{Z^2}$"

P. 9 - Line 21 from top "From Figure 3" should be "From Figure 5."

ROBERT Y. D. CHUN,[1] J.M. ASCE.—Dr. Ven Te Chow has advanced the analysis of integrating the gradually varied flow equation for prismatic channels a step forward toward simplicity and accuracy. But most important to a practicing engineer, Dr. Chow has presented a new direct method of analysis through the use of simple charts and one table. The method is accurate enough to be applied to practical problems involving channel shapes most frequently encountered, and can be used rapidly enough to be time saving.

Once a graduate student of Dr. Chow, the writer fully appreciates the masterful manner in which this broad subject is presented in a concise, orderly, and simple form. However, the writer would like to correct an error found in the general differential equation of gradually varied flow (Equation 3), and to discuss the nature of the profile of the water surface by utilizing the equation for gradually varied flow (Equation 14). Equations 3 and 14, from Proceedings Paper 838, are repeated here for convenience.

$$\frac{dy}{dx} = -\frac{S_o - S}{1 + \frac{d}{dx}\left(\frac{V^2}{2g}\right)} \tag{3}$$

1. Asst. Hydr. Engr., Div. of Water Resources, Los Angeles, Calif.

$$\frac{dy}{dx} = - S_o \frac{1 - \frac{K_n^2}{K^2}}{1 - \frac{Z_c^2}{Z^2}} \tag{14}$$

In Equation 12, Dr. Chow reveals that either the second term in the denominator of Equation 3, or Equation 12 may be incorrect. The second term in the denominator of Equation 3 is incorrect, and will be corrected herein.

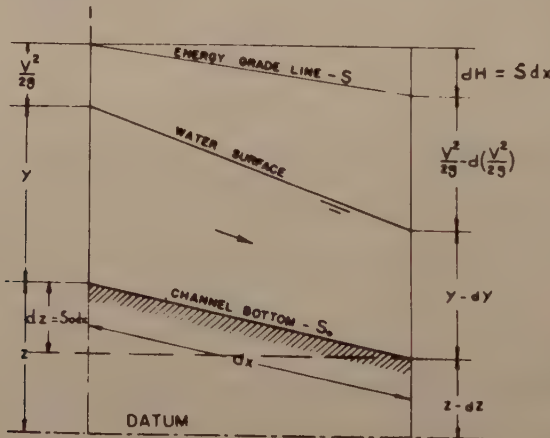

FIG 1 GRADUALLY VARIED FLOW IN A VERY SHORT REACH dx

Figure 1, as shown on page 838-28, is reproduced above with a few minor changes. Using Bernoulli's Theorem, the upstream and the downstream sections, as shown in Figure 1, were equated. After canceling, rearranging and factoring, the Equation 3-Corrected is obtained.

$$Sdx - S_o dx = dy + d\left(\frac{V^2}{2g}\right) \times \frac{dy}{dy}$$

$$\frac{dy}{dx} = - \frac{S_o - S}{1 + \frac{d}{dy}\left(\frac{V^2}{2g}\right)}$$

Nature of Profile

With only minor computation, the equation of gradually varied flow (Equation 14) can be a useful tool in determining the nature of the profile of the water surface of a gradually varied flow condition. It is a frequent occurrence that the engineer would desire only the direction of the slope of the water surface. Also, if the engineer knows the nature of the profile, he can approach the solution, with the method outlined by Dr. Chow, more intelligently. Two general conditions of flow, subcritical and supercritical, will be discussed hereinafter.

Subcritical Flow

When subcritical flow occurs, the following relationship exists:

$$y > y_c$$
$$Z > Z_c$$

However, the depth of flow can either be greater or less than the normal depth. When the depth of flow is greater than the normal depth:

$$y > y_n \text{ , then } K > K_n$$

Substituting in Equation 14: $\dfrac{dy}{dx} = - \dfrac{(+)}{(+)}$ = a negative sign

The negative value shows that the slope of the water surface is rising to the left, assuming the usual sign convention, and the direction of flow to be in the same direction as shown in Figure 1.

When the depth of flow is less than the normal depth:

$$y < y_n \text{ , then } K < K_n$$

Substituting in Equation 14: $\dfrac{dy}{dx} = - \dfrac{(-)}{(+)}$ = a positive sign

The positive sign shows that the slope of the water surface is lowering to the left.

Supercritical Flow

In the case of supercritical flow, the same procedure can be carried out as in the case of subcritical flow, and the following relationship can be shown:

With Supercritical flow: $y < y_c$
$Z < Z_c$

With depth of flow less than normal depth: $y < y_n$, then $K < K_n$

$\dfrac{dy}{dx} = - \dfrac{(-)}{(-)}$ = a negative sign

$\therefore$ The slope of water surface is rising to the left

With depth of flow greater than normal depth:

$y > y_n$, then $k > k_n$

$\dfrac{dy}{dx} = - \dfrac{(+)}{(-)}$ = a positive sign

$\therefore$ The slope of water surface is lowering to the left

Shown below is Table A giving a summary of the discussion on the nature of profile.

TABLE A

SUMMARY TABLE ON NATURE OF PROFILE, UTILIZING THE EQUATION,

$$\frac{dy}{dx} = - S_o \frac{1 - \frac{K_n^2}{K^2}}{1 - \frac{Z_c^2}{Z^2}}$$

Subcritical Flow $y > y_c$; $Z > Z_c$		Supercritical Flow $y < y_c$; $Z < Z_c$	
$y > y_n$ $K > K_n$ $\frac{dy}{dx} = (-)$ Slope rising to left	$y < y_n$ $K < K_n$ $\frac{dy}{dx} = (+)$ Slope lowering to left	$y > y_n$ $K > K_n$ $\frac{dy}{dx} = (+)$ Slope lowering to left	$y < y_n$ $K < K_n$ $\frac{dy}{dx} = (-)$ Slope rising to left

The Division of Water Resources, State of California, under the direction of Harvey O. Banks, State Engineer, and Max Bookman, Principal Hydraulic Engineer, Engineer-in-Charge of the Division's Southern California office, encourages its engineers to participate in preparation of technical papers and in its discussions. This environment is fully appreciated. Also, the assistance of Jack J. Coe, Associate Hydraulic Engineer and Robert F. Clawson, Associate Hydraulic Engineer, both of the Division of Water Resources, for reviewing and commenting on this discussion is acknowledged with thanks.

MASASHI HOM-MA.[1]—The integration method of the equation of gradually varied flow has been improved step by step by many authors. Now the problem seems to be almost perfected by Mr. Chow. Although the adaptability of the Manning formula to the flow in steep channels still remains a problem for future investigations, the other assumptions adopted by the author seem to be reasonable and have not much to be added.

Regarding the author's method, a point of theoretical interest may be added. The author's differential equation of gradually varied flow may be written as:

$$\frac{dx}{du} = - \frac{y_n}{S_o} \left[1 - \frac{1}{1-u^N} + \left(\frac{y_c}{y_n}\right)^M \frac{u^{N-M}}{1 - u^N} \right] \tag{1}$$

1. Prof., Faculty of Eng., Tokyo Univ., Bunkyo-ku, Tokyo, Japan.

Mouret[2] has pointed out that the critical slope S_c is a function of water depth and this fact causes the water surface profile to show a characteristic of flow in steep channels or even in channels of likely mild slope (Fig. 1). This characteristic is that Eq. 1 will produce a point of inflection on the surface profile in the domain of $y > y_n > y_c$.

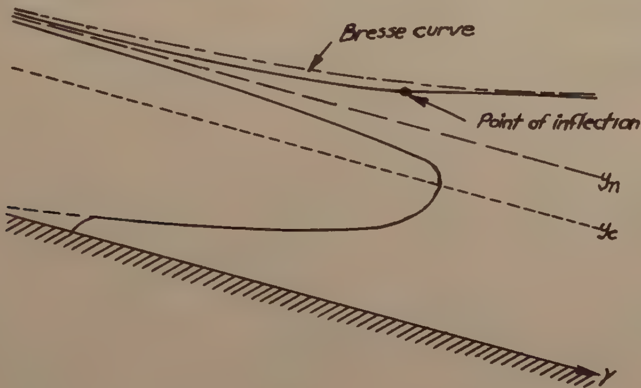

Fig. 1 Mouret's characteristic for the flow in wide rectangular channel

If N is equal to M in Eq. 1, we will get S_c = constant and such phenomenon cannot be explained analytically. The slope S_c is defined as the value of S_o which is obtained by eliminating Q from the following equations:

$$1 - [\, y_n\,(Q, S_o) \,/\, y\,]^{N} = 0 \tag{2}$$

and

$$1 - [\, y_c\,(Q, T) \,/\, y\,]^{M} = 0 \tag{3}$$

In these two equations Q is coupled with y as Q^2/y^N and Q^2/y^M respectively. Therefore, if N is equal to M, y must be eliminated with Q. Then S_c becomes independent of y and in this case the inflection point, if any, can not be determined.

Now if N is not equal to M, there may be a possibility of explaining Mouret's characteristic by the aid of numerical calculation. The condition that Eq. 1 has an inflection point in the domain of $y > y_n > y_c$, as shown in Fig. 1, requires the following equation:

$$\frac{(N - M) + Mu^N}{u^M} = N\left(\frac{y_n}{y_c}\right)^M \quad \text{at the point of inflection} \tag{4}$$

2. Mouret, "Hydraulique Generale," 1927. The writer has not seen the original book but the outline of corrections by Mouret and other authors as presented by G. Formica in "Energia Elettrica," No. 3, 1955, p. 249.

The writer has tried an approximate calculation for the example presented by the author, where N = 3.65, M = 3.43 and $y_n/y_c = 3.36/2.16$. The result indicates that the Mouret characteristic occurs at $u \doteq .185$. This condition of flow is doubtlessly out of the question for practical considerations. However, the writer is interested to see whether Mouret's characteristic has any significance to the flow in a channel of circular section. This problem perhaps can be made clear by a numerical calculation.

ALLAN NEWMAN,[1] J.M. ASCE.—Since the publication in 1932 of the Bakhmeteff(13) method for integrating the equation of gradually varied flow, there have been various modifications of his procedure in an attempt to reduce the labor involved in its application. The Engineer in attempting to evaluate these various methods is naturally led toward choosing one which has a sound theoretical basis, conforms to the physical evidences of the phenomenon it describes, and which is applied with a minimum of time consuming computation. It seems that on this basis the Bakhmeteff method supplemented by the graphs of Kirpich,(2) must be chosen. Its advantages over the Von Seggern(18) method have been previously discussed by Kirpich and will not be repeated here. The author's work has succeeded in eliminating the additional table required by the Von Seggern method, but at the expense of introducing certain computational complications. A comparison of the Bakhmeteff equation for gradually varied flow with that of the author illustrates this.

The Bahkmeteff equation is:

$$L = \frac{y_n}{S_o}\left\{(u_1 - u_2) - \left(1 - \frac{S_o}{\sigma}\right)\left[F(u_1, N) - F(u_2, N)\right]\right\}$$

where σ (sigma) the critical slope and N the hydraulic exponent are obtained from the Kirpich graphs by computing $\frac{y}{b}$ and $\left(\frac{1.486}{n}\right)^2 b^{1/3}$.

The author's equation is:

$$L = \frac{y_n}{S_o}\left\{(u_1 - u_2) - \left[F(u_1, N) - F(u_2, N)\right] + \left(\frac{y_c}{y_n}\right)^M \left(\frac{J}{N}\right)\left[F(u_1^J, J) - F(u_2^J, J)\right]\right\}$$

where M, the exponent for the section factor, and N the hydraulic exponent is obtained graphically by computing $\frac{y}{b}$, J equals $\frac{N}{N-M+1}$ and the value of y_c is obtained graphically by computing $\frac{Q}{\sqrt{g}\, b^{2.5}}$

It is clear that the complicated third term of the author's equation may be replaced by multiplying the second term by the simple factor $\left(1 - \frac{S_o}{\sigma}\right)$. In fact

1. Project Engr., Andrews, Clark and Buckley, Cons. Engrs., New York, N. Y.

by noting that, $\sigma = \frac{Z^2 g}{K^2}$, it can be shown, by suitable arithmetic procedures, that the simple Bakhmeteff equation forms an identity with the author's equation.

While in some applications values of σ must be obtained by steps, these would be no more numerous over a given reach, than those required for the evaluation of the author's exponent M, inasmuch as an error in this factor is compounded by being in effect multiplied by itself since it also controls the values of $\left(\frac{y_c}{y_n}\right)^M$, J and $F(u^J, J)$. Also, the value of N in the Bakhmeteff equation for practical purposes need only be evaluated to the nearest tenth which is more consistent with graphical accuracy, while the author abstracts the values of N, and M, to hundredths from his graphs, a procedure which is probably required by his method. Finally, while some of these difficulties are apparent in the author's comparatively simple illustrative problem, solutions by the author's method of the somewhat more complex problem of computing a discharge curve (Von Seggern(18) ex. 7) or of the problem of obtaining water surface elevations at specific distances from the control depth, would show how onerous the computations become, as compared with solution by the Bakhmeteff equation.

Two aspects of the paper deserve commendation.

First, the Bakhmeteff table of varied flow functions is now published as a Proceedings Separate making it more generally available, and secondly a brief survey of the various existing methods of computing water surface curves has been included, giving the casual reader a broader perspective into the subject background without requiring the investigation of numerous references.

STEPONAS KOLUPAILA.[1]—Author is to be acclaimed for his ingenious effort to find a correct and lucid solution for a complicate problem of nonuniform flow in prismatic channels. An excellent idea to confine the computation to a single function was crowned with success. This varied-flow function was originated by B. A. Bakhmeteff; tables provided for these functions proved to be too short, thus the author put enormous amount of work in expansion of these tables. This is his particular achievement.

The use of a single table is an answer to the problem, as suggested by the writer in his discussion to the paper by M. E. Von Seggern (Transactions ASCE, 115(1950), p. 95).

It is regrettable, however, that author deliberately disregarded the fact of nonuniform velocity distribution across a current and neglected so-called Coriolis' correction factor. So did Bakhmeteff, although he had agreed to change this in a new edition.

Writer believes that this new excellent method will find wide appreciation among hydraulic engineers.

The summary of existing methods (table 1) could be enlarged by several more, as by H. Walther (1903), N. N. Pavlovskii (1924), N. Westerberg (1924), I. I. Levi (1928), A. N. Rachmanov (1930), R. R. Chugaiev (1931), M. D. Chertousov (1934), P. W. Werner (1940), A. Vitols (1942), I. I. Agroskin

1. Prof. of Civ. Eng., Univ. of Notre Dame, Notre Dame, Ind.

(1944), R. B. Jansen (1951), T. G. Voinich (1953), etc.

Writer would like to improve one historical detail. The tables for use of Bakhmeteff method were not lost in the turmoil of revolution and civil war: they were copied in hand-written form, until they had been added to a second Russian edition of the book in 1928. These tables had no typographic errors; exponent were different (2.0, 2.5, 3.0, 3.25, 3.5, 3.75, 4.0, 4.5, 5.0, and 5.5) and the values of η were more detailed. These tables had been reprinted many times in Russian textbooks and manuals (not mentioning the name of the author anymore; such plagiarism can be explained by amnesia).

CORRECTIONS.—

p. 838-7, line 3 from the top, "Let $v = u^J$" should be "Let $v = u^{N/J}$."

p. 838-7, line 1 below Eq. (37), "in which $u = y/y_n$, $v = u^J$" should be "in which $u = y/y_n$, $v = u^{N/J}$."

p. 838-10, line 8 from the top, "and $v = u^J$" should be "and $v = u^{N/J}$."

p. 838-10, the 3rd and 5th columns of v and F(v,J) in the table under step 4) of the example should be corrected as

v	F(v,J)
1.625	0.213
1.015	1.293
	-1.080

p. 838-10, the equation under step 6) of the example should be corrected as

$$L = 2{,}100\ [0.476 - (-0.877) + (0.180)(-1.080)] = 2{,}430 \text{ ft.}$$

p. 838-13, line 11 from the bottom, "$v = u^J$" should be "$v = u^{N/J}$."

p. 838-25, line 1 from the top, "(p. 10 or 12)" should be "(p. 10 of 12)."

p. 838-27, line 1 from the top, "(p. 12 or 12)" should be "(p. 12 of 12)."

Discussion of "RAINFALL DEPTH-DURATION RELATIONSHIPS"

by Herbert M. Corn
(Proc. Paper 840)

D. M. HERSHFIELD,[1] and W. T. WILSON,[2] A.M. ASCE.—The author's problem evidently was to synthesize the rainfall-intensity regime for a remote region lacking short-duration data. In the absence of local data, United States data were used (Yarnell's tabulation of each station's most intense rainstorm as of 1933).

A question that immediately arises is: how analogous is the rainfall regime of the remote area to that of the area from which the author selected his 60 stations? The United States storms studied were mostly summer storms and, as the author states, were generally convective. A convenient measure of the degree to which convective storms typify a region's climate is the incidence of thunderstorms. In the central United States, the region from which the 60 storms apparently were selected, the mean annual number of thunderstorm days varies from about 30 to 70, with an average of about 50.[3] Examination of climatic records for French Morocco shows the mean annual number of thunderstorm days to vary from about 2 to 20, with an average of about 8.

The moisture source region for intense storms in the central United States is the South Atlantic and Gulf of Mexico, where surface dewpoints approaching 80°F ensure a high moisture charge. In French Morocco, as in California, the moisture source region is the relatively cold water to the west of the coast, where dewpoints rarely exceed 60°F—a much lower moisture charge.

The 2-year 24-hour precipitation in the central United States varies from about 2 to 5 inches, whereas in French Morocco less than 10 percent of the stations report a 2-year 24-hour value in excess of 2 inches. Figure 7, which shows the relation of hourly to 24-hour rainfall as a function of thunderstorm experience, is based on northern United States stations for estimating short-duration rainfall in remote cold regions.[4]

Another question that deserves consideration is: what basis is there for extending the generalized depth-duration curve beyond the range of the data—say, from 80 minutes to 12 or 24 hours? The curve shown in figures 3 and 4

1. Meteorologist, Cooperative Studies Section, Weather Bureau, U. S. Dept. of Commerce, Washington 25, D. C.
2. Chief, Cooperative Studies Section, Weather Bureau, U. S. Dept. of Commerce, Washington 25, D. C.
3. U. S. Weather Bureau, Dept. of Commerce. Weather Bureau Technical Paper No. 19. Mean number of thunderstorm days in the United States. Washington, D. C. December 1952.
4. U. S. Weather Bureau, Dept. of Commerce. Rainfall intensities for local drainage design in Arctic and Subarctic regions of Alaska, Canada, Greenland, and Iceland. Washington, D. C. September 1955, Figure 1.

is based on short-duration rainfall; in spite of the 'Introduction's' implication of its application to estimating short-duration rainfall from 12-hourly data from remote areas, and in spite of the 'indications' suggested in the 'Conclusion,' the writers question any basis for extrapolating the curve to durations of 12- or more hours.

The mass curve of rainfall for most days of appreciable rain, or even for most storms, is typified by periods of relatively intense rainfall, with little or no rainfall the rest of the time. These periods of intense rainfall are known to meteorologists as "burst," and as Corn has shown by his analysis of Yarnell's data they tend to have a typical and continuous distribution of rainfall increments. The writers see no reason to assume that the "within burst" distribution would be anything like the "among burst" distribution. Among the bursts, according to analysis of mass curves of daily rainfall of, say 2 inches or more, the periods of no rainfall typically occupy nearly half the day. Exceptions to this include hurricane rainfall. Attempts to generalize on the typical distribution would have to recognize seasonal, regional and other differences.

The curve of figures 3 and 4 says, as we understand it, that the ratio of 5- to 120-minute rainfall is the same as the ratio of the 1- to 24-hour rainfall, and that this ratio has general application. The ratio of the 1- to 24-hour rainfall from this curve is about 10 percent, whereas this ratio varies from about 30 percent to 60 percent in the central United States and about 10 percent to 70 percent for the entire United States. Figure 8 shows a regional difference in the 10- to 60-minute rainfall between coastal and interior zones of western United States.[5] This regional difference is related to average hourly rainfall amounts. Figure 9[6] is based on Yarnell's and other data and shows that the ratio of 10- to 60-minute rainfall averages about 0.55 for total hourly rainfall of 1.0 inch to 2.0 inches, and about 0.30 for total hourly rainfall of 4.0 inches to 5.0 inches.

To illustrate the limitation of extrapolating such a curve beyond the range of data, we can use the Lexington, Kentucky storm of July 3, 1931, which is described in figure 5 and table 2. The suggested method has been applied to a total duration of 50 minutes, and is shown to work well. It fails when applied to the 24-hour period, which has a total depth of 3.57 inches for 24 hours. An hour is roughly 4 percent of a day; entering figure 4 with a duration of 4 percent, the corresponding rainfall is about 10 percent of the daily depth. Ten percent of the daily total, 3.57 inches, is .36 inch, whereas the observed value is about 8 times that much. This difficulty can be avoided in part if the duration of actual rainfall for the reporting period of 24 hours (or 12 hours) is known. This information is seldom available, however, except at stations having continuous observation of the weather. Also, the sequence

5. U. S. Weather Bureau, Dept. of Commerce. Weather Bureau Technical Paper No. 24. Rainfall intensities for local drainage design in the United States for durations of 5 to 240 minutes and 2-, 5-, and 10-year return periods. Part I: West of 115th meridian. Washington, D. C. 1953, Figure 6.
6. U. S. Weather Bureau, Dept. of Commerce, and Corps of Engineers, War Department; Hydrometeorological Report No. 5, Thunderstorm Rainfall, Vicksburg, Mississippi, 1947, Figure 141.

of rainfall increments is rarely available at stations where a synthesis of the regime would be pertinent.

Another feature of this curve should be noted. The coordinates of figure 2 are cumulative precipitation and cumulative time, both expressed in percent of total; the points are distributed quite closely around a simple curve on semi-logarithmic paper. This good fit will also be the case when two series of random numbers, with the same operations performed on them, are plotted on either logarithmic or semi-logarithmic paper.

The author briefly discusses the lack of similarity in the measurements for stations of rather close proximity and concludes that these differences are a result of poor observational practices. Convective precipitation is naturally spotty and it is not unusual for the heaviest precipitation to be concentrated in a small area. Whatever dissimilarity in gage measurements appears may well be a function of convective rainfall pattern rather than an indication of the quality of the observations.

An examination of the Burlington, Vermont, maximum annual rainfall values for durations of 5 minutes to 24 hours for 50 years of record reveals that it is not unusual for 75 percent of the rainfall in these storms to occur in about 10 percent of the storm time. What the author refers to as "peculiar" is simply a characteristic of rainfall regime for Burlington. In fact, this situation occurs, on the average, about once every three years.

In conclusion, the writers heartily endorse Mr. Corn's statement that more research is needed, and his hope that continued cooperation of hydrologists and meteorologists will produce the desired answers.

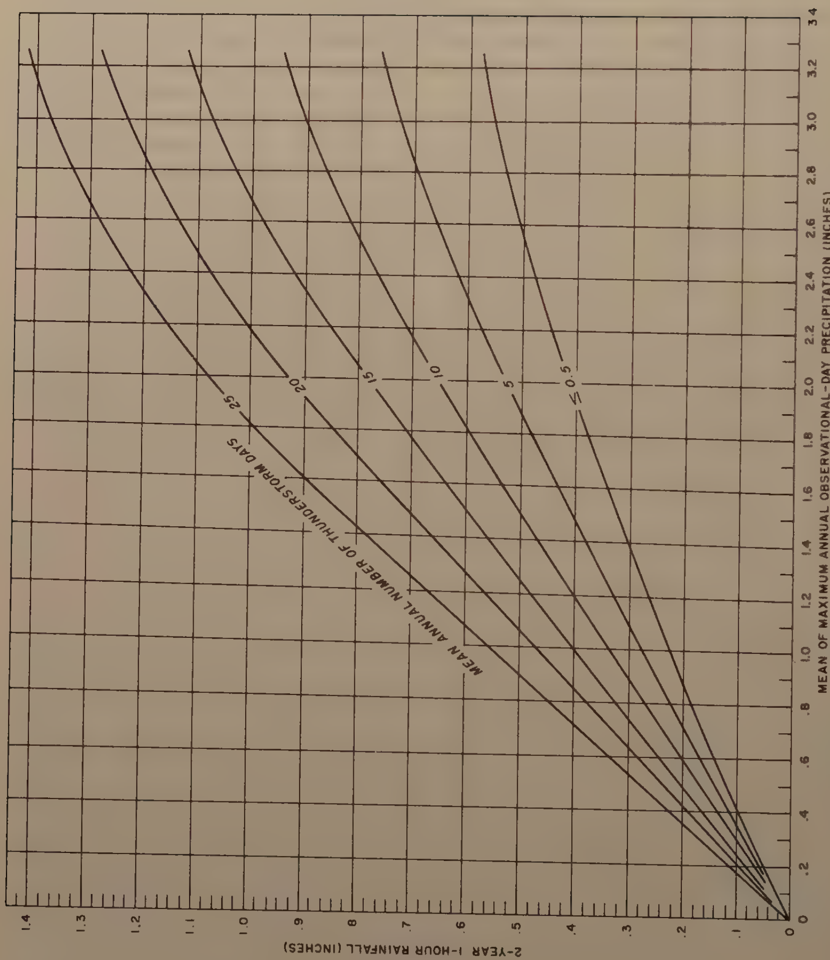

FIGURE 7 DIAGRAM FOR ESTIMATING 2-YEAR 1-HOUR RAINFALL

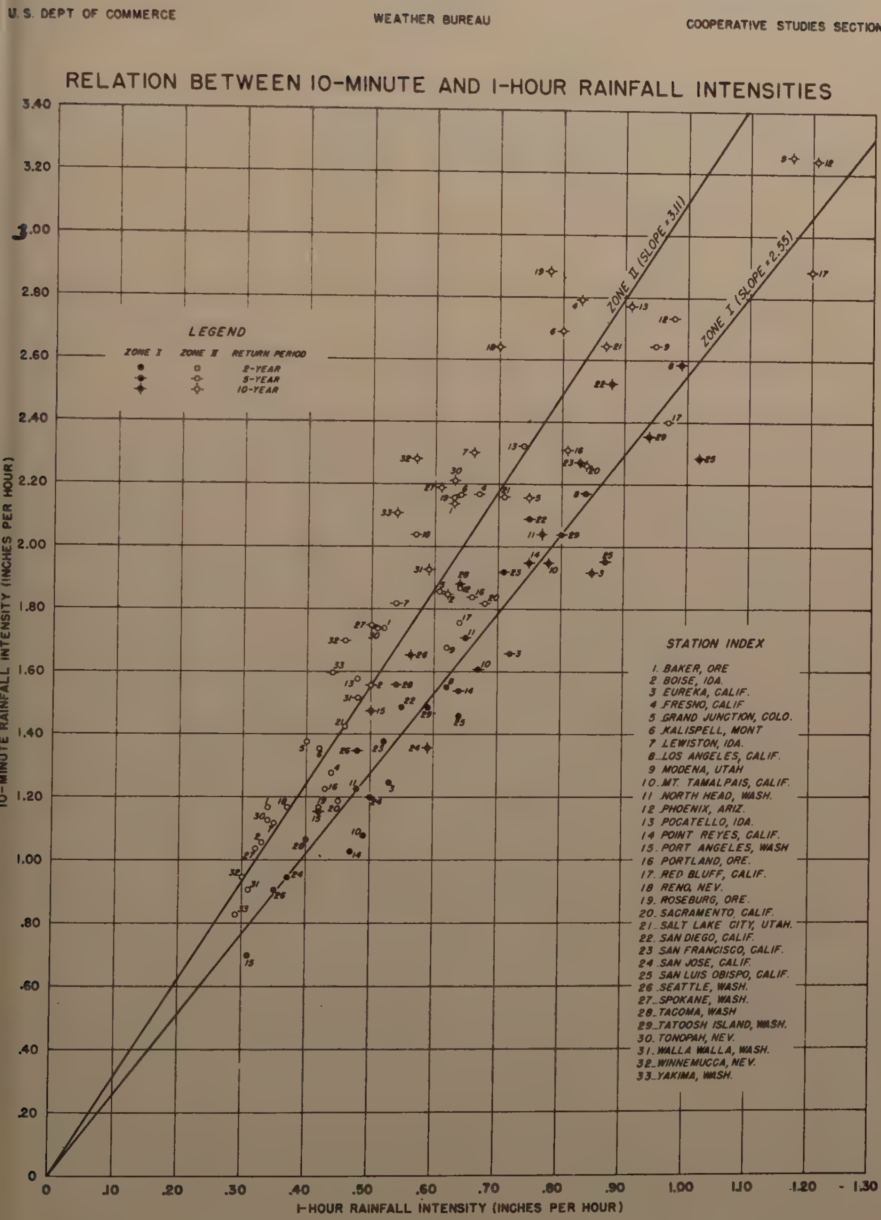
U. S. DEPT OF COMMERCE
WEATHER BUREAU
COOPERATIVE STUDIES SECTION
RELATION BETWEEN 10-MINUTE AND 1-HOUR RAINFALL INTENSITIES
10-MINUTE RAINFALL INTENSITY (INCHES PER HOUR)
1-HOUR RAINFALL INTENSITY (INCHES PER HOUR)
ZONE II (SLOPE = 3.11)
ZONE I (SLOPE = 2.55)
LEGEND
ZONE I
ZONE II
RETURN PERIOD
2-YEAR
5-YEAR
10-YEAR
STATION INDEX
1. BAKER, ORE
2. BOISE, IDA.
3. EUREKA, CALIF.
4. FRESNO, CALIF
5. GRAND JUNCTION, COLO.
6. KALISPELL, MONT
7. LEWISTON, IDA.
8. LOS ANGELES, CALIF.
9. MODENA, UTAH
10. MT. TAMALPAIS, CALIF.
11. NORTH HEAD, WASH.
12. PHOENIX, ARIZ.
13. POCATELLO, IDA.
14. POINT REYES, CALIF.
15. PORT ANGELES, WASH
16. PORTLAND, ORE.
17. RED BLUFF, CALIF.
18. RENO, NEV.
19. ROSEBURG, ORE.
20. SACRAMENTO, CALIF.
21. SALT LAKE CITY, UTAH.
22. SAN DIEGO, CALIF.
23. SAN FRANCISCO, CALIF.
24. SAN JOSE, CALIF.
25. SAN LUIS OBISPO, CALIF.
26. SEATTLE, WASH.
27. SPOKANE, WASH.
28. TACOMA, WASH
29. TATOOSH ISLAND, WASH.
30. TONOPAH, NEV.
31. WALLA WALLA, WASH.
32. WINNEMUCCA, NEV.
33. YAKIMA, WASH.

Figure 8

U. S. DEPARTMENT OF COMMERCE WEATHER BUREAU HYDROMETEOROLOGICAL SECTION

TYPICAL MASS CURVES OF ONE - HOUR THUNDERSTORM RAINFALL

(Point Rainfall — Fixed Station)

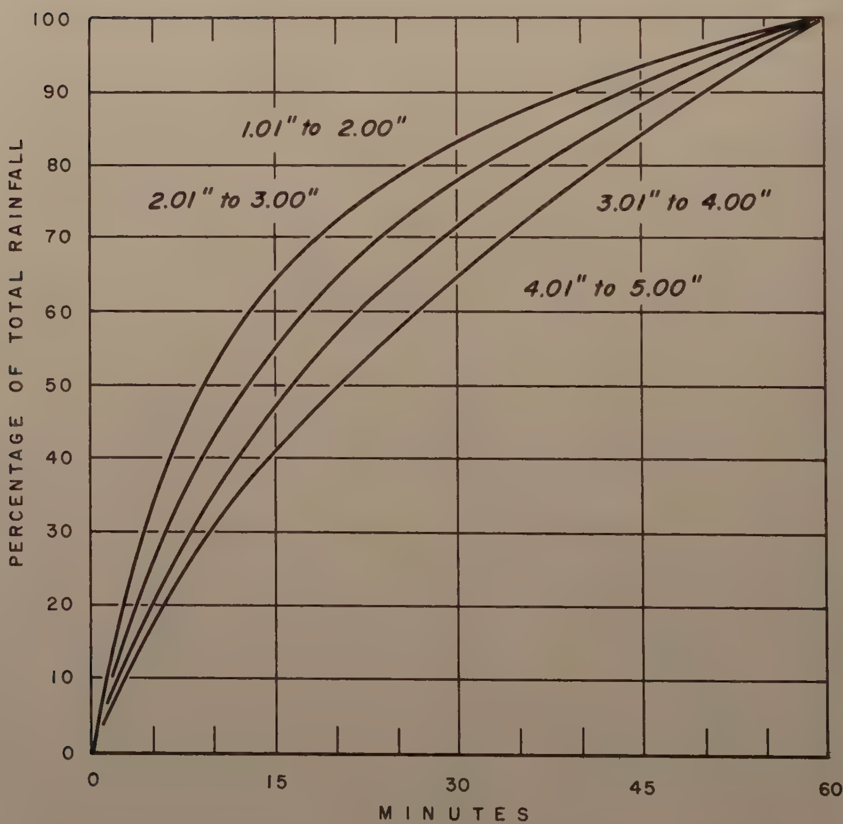

FIGURE 9

JOURNAL
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

GEOLOGY OF SOME AMERICAN ESTUARINE HARBORS

Parker D. Trask[1]
(Proc. Paper 956)

ABSTRACT

The essential requirements of harbors are protection from wave action, space for maneuvering of ships, and adequate depth of water. The principal causes of natural bodies of water suitable for harbors are (1) rise of sea level which causes the sea to inundate previously-formed valleys or low places; (2) deformation of the land, causing the land to sink so that the sea occupies the depressions; (3) differential erosion, which causes the soft rocks to be washed away, leaving low areas where the streams flow and valleys form, so that when the sea level rises or the land sinks, water can come into the low-lying areas; (4) near-shore deposition of bars which form barriers to bays, thus protecting them from waves; (5) glacial action which either scours out a basin which subsequently fills with water or deposits morainal and other material, forming islands or bars that protect the harbor from waves; and (6) mouths of large and deep rivers.

Examples of typical harbors developed by these processes are Boston, formed by glacial action and depression of the land; New York, Philadelphia, Norfolk, Savannah, Charleston, and San Diego, formed by drowning of rivers either by rise of sea level or lowering of the land; New Orleans and Portland, near the mouths of large rivers; Galveston, by the formation of barrier bars and widening of the sides by wind action; and San Francisco and Seattle by deformation and lowering of the earth's crust. All these harbors except New Orleans and Portland show the effects of differential erosion.

Factors that affect silting and shoaling of harbors are: distribution of currents; quantity, size-distribution, and mineral composition of sediment; amount and rate at which water is brought down by streams; salinity distribution; type of clay minerals; and amount of organic matter and sewage in the water.

Note: Discussion open until September 1, 1956. Paper 956 is part of the copyrighted Journal of the Hydraulics Division of the American Society of Civil Engineers, Vol. 82, No. HY 2, April, 1956.

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INTRODUCTION

The control of harbors is a difficult problem. Two essential requirements of harbors are: (1) location in a place protected from wave action, and (2) adequate depth of water. The distribution of depth of water in natural harbors represents equilibrium conditions with the environmental factors affecting the area. If man modifies the harbor, the water and sediments attempt to reach equilibrium in light of the new conditions, frequently much to his discomfort, as witnessed by the more than 100 million yards of material that must be dredged annually from the harbors in the United States to keep them open.

The Committee on Tidal Hydraulics of the Corps of Engineers, under the leadership of Clarence F. Wicker and J. B. Tiffany, for several years has been studying the environmental processes that affect depth of water in harbors. These factors have been well-expressed by Wicker and Rosenzweig (1950) in the first published report of this committee. The present symposium, of which this paper is a part, summarizes the progress that has been made by the committee since 1950. This paper describes the geologic causes of harbors and the environmental factors that affect the character of estuarine deposits in the United States. It also contains a brief description of the geologic cause of typical American harbors.

Geologic Factors Affecting Harbors

The principal geologic factors affecting estuaries or harbors are: (1) the rate of change of sea level, (2) the deformation of the land, (3) the climate or rate of change of the climate, (4) the nature and distribution of the bed rock in the area, and (5) the inherited configuration of the land resulting from its previous geologic history. The soil, which is one of the principal sources of the sediments deposited in the harbor, is also a result of these environmental factors, as Jenny (1950) has shown so clearly. Soil depends primarily upon the climate, bed rock, and topography or slope of the land. Man, in his efforts to cultivate the land, upsets the processes which produced the previous equilibrium or near equilibrium. The chief difference caused by man is acceleration of erosion and greater supply of sediment to the harbor, which results in shoaling problems of concern to the community. Man also, in deepening and straightening channels and in building breakwaters and jetties, interferes with the natural circulation, thus resulting in the deposition and scour of sediment in unwanted places.

The geologic factors that influence the deposition and character of sediments are essentially the same factors as the hydraulic engineer recognizes. The geologist, however, thinks primarily in terms of the present state, and of the rate of change, of the environment. Four fundamental factors control environment. These factors are climate, configuration of land, nature of bedrock, and past geologic history. The sediment and soils in individual environments strive to attain, or have already attained, equilibrium with their environment. If geologic events, or man, cause the environment to change, the old equilibrium is upset and the area tries to come into equilibrium with the new conditions. Thus, in areas in which geologic conditions or climate are, or have been recently, changing at a relatively rapid rate, the sediments and depth of water are likely to be different than they were formerly.

As the factors causing shoaling in harbors are discussed by other papers in the present symposium, they will not be described at length here. The

principal factors are current distribution; quantity, size distribution, and mineral composition of sediment; amount and rate at which water is brought down by streams and in by tides; the distribution of salinity; and the addition of organic matter in the form of sewage to the harbor. The type of clay minerals is a very important factor affecting deposition, but it is a result of a combination of the basic environmental factors just mentioned. The type of clay brought down by the stream materially influences the sediments deposited in the harbor. The presence of water of different salinity or ionic concentration in the harbor causes changes in the clay minerals (Grim, 1953).

As clays have only just begun to be studied intensively by geologists, exact conclusions as to what happens or may happen have not yet been reached. Certainly ions are exchanged in the clay molecules, causing changes in the physical and chemical behavior of the sediments. New clay minerals also may be formed. Shepard and his associates (1953, 1954) have found that on the continental shelf off Southern Texas, chlorite and glauconite take the place of common clay minerals brought down by the rivers. Recent work also has shown that organic matter forms compounds with the clay minerals (Grim, 1950, pp. 265-269). That is, the organic matter ties into the structure of the clay and changes significantly its properties. Such reactions between organic matter and clay, aided probably by the effect of bacteria, perhaps are one of the causes of the serious silting below Philadelphia. In this area more than 15 million cubic yards of material must be removed annually. Harbor engineers would do well to give serious attention to the problem of clay mineralogy with respect to silting of harbors.

Effect of Change of Sea Level

The principal geologic cause of harbors in the United States is the recent world-wide rise of sea level (Marmer, 1948); (Russell, 1952); (Shepard, 1948). Geologists are generally agreed that, not so long ago geologically speaking, sea level was lowered at least 200 feet and probably as much as 300 feet, and perhaps more, all over the world. The cause of this lowering of sea level is ascribed to the removal of water from the ocean to form the great glaciers that covered the polar regions of the world during the past ice age. These ice caps began to melt at a rapid rate 10,000 to 20,000 years, or perhaps longer ago. This melting has resulted in a gradual rise of sea level. The rise depended upon the vagaries of climate, and has proceeded at a variable rate. According to Disney (1955), sea level along the coasts of the United States has risen about half a foot in the last 50 years. It is not at all certain, however, that the rate was so rapid before that time. The distribution of ancient cities around the world suggests that sea level may not have changed appreciably during the last 2000 years. Nevertheless, there is little doubt but that sometime before the time of Christ sea level had risen materially from its previous low stage. As a result, the sea invaded the land and drowned the valleys of streams that flowed out to the ocean during glacial time. As sea level rose, silt was deposited in the drowned valleys. In places, particularly on the Gulf Coast, silting has proceeded almost as rapidly as rising of the sea (Thompson, 1955); in other places, such as Chesapeake and Delaware bays, sedimentation has not kept pace with rising level, with the result that long estuaries have formed.

Deformation of Land

In addition to rise of sea level, the land itself in places has been deformed in relatively recent geologic time. The coast line near Seattle, San Francisco, and Boston has been depressed and broken by earth-deforming movements. This deformation has proceeded much more rapidly than the addition of sediment, with the result that deep bays now exist in these regions.

In other areas, such as the Atlantic and Gulf coasts, the land has been slowly tilted or depressed seaward. This tilting of the land resulted in down-cutting of the valleys in such a way that the present streams flow at lower elevation with respect to the adjoining land than in earlier geologic time. In places the axis of tilt is landward from shore with the result that the land seaward from the axis of tilt has been depressed below sea level. As this tilting began a long time ago, areas formerly occupied by sea have since been filled with sediment. An example of such warping is the Mississippi embayment shown in Fig. 1. This embayment has since been blanketed with sediments of Tertiary and Quaternary age.

In the areas formerly occupied by ice, north of a line extending northwestward from New York City to the Great Lakes and beyond, the land has been elevated as much as 700 feet since the retreat of the glaciers. The southernmost limit of tilting and the extent of the ice are shown upon Fig. 1 (Leverett and Taylor, 1915); (Flint, 1948). This tilting has resulted in depth changes along the New England Coast. The uplift caused by this tilting tends to lessen the effect of the previous deformation of the coast in that area.

The warping of the continent has taken place in steps. Long periods of quiet have interrupted the deformation of the land. During these periods of quiet the streams have moved back and forth across their valleys and carved more or less broad flood plains. When the land began to move upward again, the streams cut new trenches in the old flood plains. The end result is a series of terraces. Most of these terraces are above sea level, but Chesapeake and Potomac bays have a pronounced terrace whose surface is 10 to 15 feet beneath the present water level. Either this terrace was cut during the ice age when sea level was lower, or the land has been tilted to such an extent that the former terrace was depressed below sea level.

Differential Erosion

The hardness of the underlying bed rock also influences the processes of erosion and the resulting configuration of land. If the rocks are hard they do not erode; if they are soft they wear away readily. As a result, valleys form where the rocks are soft. Hills or headlands are produced where the rocks are hard. Consequently, harbors or estuaries generally are found where the rocks are soft. If the rocks and sediments that form the borders of the estuaries or harbors are poorly consolidated or cemented, such as on the Gulf Coast, they erode readily by waves. As a result the harbors increase in size. The banks of Galveston Bay are estimated to be eroding at the rate of three feet a year (U.S. Corps of Engineers, 1942, p. 61). The borders of Chesapeake and other bays similarly are being eroded by the wind and waves, with the result that these bays too are increasing in size.

The problem then arises—what happens to the silt that is eroded from the banks? Is it transported farther out into the bay and deposited in shoals, or is it carried out to sea? According to Thompson (1953), many bays in the

Gulf Coast maintain a constant depth depending upon the wave size, but Shepard (1953) cites data to show that other bays are silting up. Thompson points out that the larger bays are deeper than the smaller bays because they have greater fetch for the generation of waves. As a consequence, wave action extends to a greater depth and stirs up the bottom. The depth of these bays thus represents an equilibrium condition in which any excess sediment that normally would be deposited is agitated by wave action and kept in suspension until such time as it is transported out to the ocean. Shepard estimates that the Trinity River in Texas transports five times as much sediment through Galveston Bay as is deposited in the bay. That is, more than 80 per cent of the sediment by-passes the bay and eventually reaches the sea.

The problem, however, is not so simple as just outlined. Much depends upon the size of the particles that are transported to the bay or are already on the bottom of the bay. The sediments in the Gulf-Coast bays are fine-grained clays which easily stay in suspension once they are agitated. The sand and silt brought down by the Trinity River into Galveston Bay, as Shepard has shown, have shoaled the water at the entrance of the river into the bay by as much as 5 feet in the past 100 years. It is also conceivable that a river could bring down so much fine silt and clay that the waves could not transport away all that was brought down, even though the material was readily transportable by the water in the bay.

Effect of Currents

Currents also affect silting in harbors. If currents are directed seaward, more sediment can be removed than if they are in some other direction. Longshore currents may produce barrier bars, which cause bays to form behind them; as for example, Pamlico Sound on the Atlantic Coast and Galveston Bay on the Gulf Coast. These bars generally produce smooth arcuate islands or spits parallel to the coast, leaving marshland and estuaries behind them with more or less narrow outlets. Bars are more common on drowned than on rising coasts. The California Coast has few barrier bars compared with the Atlantic Coast.

Glacial Action

Continental glaciers also profoundly modify harbors. The ice continually moves outward from centers of accumulation. As the ice migrates it picks up rock and silt. This debris is deposited when the ice melts. At the front of the glaciers, determined by the position where the rate of melting equals the rate of advance of the ice, large masses of debris called terminal moraines are formed. The rock material that is continually brought along by the glaciers is dumped as the ice melts. These terminal moraines, when located along the coast, may form islands which cause good harbors. Boston Harbor is an example.

Also when the glacier eventually melts, all the debris which is contained within it is left behind upon the ground as a blanket. This morainal material varies in composition from huge blocks many feet in diameter to silt and clay. Boston Harbor and Puget Sound were covered with ice which dropped a great variety of material on the bottom of the water. In Boston Harbor great variation, from silt and clay to gravel, is found throughout the harbor. Also the depth of water varies, depending upon the amount of debris deposited. So

much material has been laid down in some places that shoals or isles were formed. Puget Sound is so deep that the glacial material deposited in it has caused few problems of navigation.

Geologic Description of Harbors

Boston Harbor

Boston Harbor is of glacial and deformational origin (Currier, 1952; Shaler, 1893). The New England Coast has been broken and depressed beneath the surface in past geologic time. Following this action, the area was covered by thick sheets of ice, which when they melted left behind a more or less thick mass of morainal material on the ground and off the coast. Most of the sediment now found in Boston Harbor consists of this morainal material. Sediment size ranges from clay to huge boulders. Much of the clay presumably represents recent deposition, but most of the harbor bottom is just what the glacier happened to leave. The depth of water is variable, owing to the vagaries of glacial deposition.

The subsurface sediments, as indicated in Fig. 2, consist of sand, silt, and clay of variable thickness, generally 20 to 30 feet. In places these sediments rest upon crystalline bedrock, in other places they rest upon blue clay, as a rule 40 to 60 feet thick. This clay locally has been oxidized to form yellow clay. Interspersed in some of the deposits are peat and organic clay.

New York Harbor

New York Harbor is essentially a drowned river. The Hudson River extends seaward into a deep submarine canyon that crosses the continental shelf in a trench (Veatch and Smith, 1939). The canyon can be traced to a depth of more than 10,000 feet below sea level. In the vicinity of New York City, the presence of this canyon represents a former submergence of the land. Following that period of lowering, a series of silts and sandy silts were deposited. Subsequently, during the last glacial period when sea level was again lowered, the river cut a valley in these sands and silts. Since that time this glacial valley has been filled with sand and silt brought down the river. The depth of the ancient gorge of the Hudson River at New York is at least 280 feet (Fig. 2), and drill holes up the river have gone more than 400 feet without striking bedrock (Fluhr, 1953). Long Island is covered by the end moraine of a big ice sheet. Long Island Sound occupies the low ground behind the moraine. Manhattan is composed of crystalline rocks.

Philadelphia Harbor

The harbors south of New York are limited landward by what is called the Fall Line. A line extending more than 200 miles southward from New York through Trenton, Philadelphia, Baltimore and Washington separates the soft Tertiary and Cretaceous sediments of the Atlantic Coastal Plain from the crystalline and other rocks of the Piedmont area to the west. Falls in the river at the contact between the crystalline and sedimentary rocks terminate navigation. The trace of this "Fall Line" and the distribution of crystalline and sedimentary rocks along the Atlantic Coast is shown upon Figure 1.

Philadelphia is near the headwaters of Delaware Bay. This bay represents the drowned valley of the Delaware River. Depth to sedimentary bedrock in places is more than 200 feet, and depth to the crystalline rocks is several hundred feet more. The old valley clearly was formed when the land stood considerably higher than now. Since the time of formation of the valley, the

land has sunk and the valley filled with sediment. Subsequently, the valley was again deepened, during the glacial lowering of sea level. It has since been largely filled with material brought down by the Delaware and Schuylkill rivers. The silting problem in the harbor is complicated by the addition of finely powdered coal from the anthracite mines up the Schuylkill River and by the presence of organic clay deposits, which cause a serious dredging problem downstream from Philadelphia.

Norfolk Harbor

Norfolk Harbor is at the mouth of the Chesapeake and Potomac estuaries. These estuaries are drowned valleys which have not yet filled with sediment since the glacial lowering of sea level. The sediments are mainly silt and clay.

Savannah and Charleston Harbors

The rivers on the southern Atlantic Coastal Plain have not been depressed below sea level as much as the northern rivers. The harbors at Savannah and Charleston appear more to be the result of post-glacial rise of sea level than of downwarping of the Atlantic Coast, though downwarping cannot be positively excluded. The problems in these harbors are mainly those caused by man when he tries to change nature's equilibrium by deepening the water. Before man came, a natural depth of 10 to 15 feet prevailed. When these depths were exceeded, nature tended to fill in the dredged areas in places where currents were weak.

The sediments consist of silt, clay and silty sand. The river brings down much silt and clay; a considerable proportion of the sand is brought in from the ocean by tidal currents, as is attested by the presence of hornblende in the beach sands and in the sand in the harbor, compared with its scarcity or absence in the sediments brought down by the streams entering the harbor (Schultz, 1952). The distribution of hornblende indicates that much of the sand is transported 5 miles, and some of it as much as 10 miles, upstream from the ocean. As shown by Figure 2, the subsurface deposits consist of 10 to 60 feet of sand, gravel, and shells, presumably in part representing material filled in during the post-glacial rise in sea level. Below this are several hundred feet of blue clay, shale, sandstone, and other sediments of Tertiary and Cretaceous ages. Similar subsurface conditions are found at Savannah.

New Orleans Harbor

The harbor at New Orleans is on the Mississippi River. As the harbor lies one hundred miles upstream from the mouth of the river, it is hardly an estuarine harbor, though salt water is now wedging a long way up the river. Under normal conditions the water ranges in depth between 35 and 100 feet from New Orleans to the principal mouths at Southwest and South passes. The volume of water coming down the river and going out Southwest Pass is sufficient to keep the sediment in suspension to considerable depths. The depth of water in the other channels is less. The chief problem of maintaining the channel to specified depths is to maintain adequate flow in the river and to control the outlets by proper jetties (Holle, 1951).

The geologic history of the delta has been well described by Russell (1952), Morgan (1952), Fish (1952), and Scruton (1955). The sediments on the delta are very fine sand, silt, and clay. According to Holle (1951, p. 124), the river transports some 500 million tons of sediment annually. This sediment

is two per cent very fine sand (62 to 125 microns); 48 per cent silt (4 to 62 microns); and 50 per cent clay (less than 4 microns). As in other harbors, silting is a more serious problem in the small channels than in the large channels. At New Orleans the subsurface sediments consist of 100 or more feet of sand, silt, and clay, below which are Pleistocene deposits, in part of oxidized clay (Fig. 2). These clays have been downwarped in relatively recent geologic time.

Galveston Harbor

Galveston Bay is typical of the barrier bays along the Gulf Coast. It is an elliptical bay, 20 miles in maximum diameter, protected from the ocean by a barrier bar composed of fine sand. The Trinity River, one of the major Texas streams, flows into the northwest part of the bay. The average depth of the bay is 7 to 9 feet. According to Shepard (1953) the bay has shoaled about two feet in the past century, and as much as five feet on the delta of the Trinity River. Shepard also states that more than 80 per cent of the silt brought into the bay is transported out to sea, and less than 20 per cent is deposited on the bottom. The sediments of the bay consist of mud and sandy silt to a depth of 50 to 100 feet, below which is firm clay of Pleistocene age. The surface of the Pleistocene clay slopes seaward gently. It is encountered at greater depth at the south than at the north end of the bay.

San Diego Harbor

The harbors on the West Coast are different from those on the East Coast. The United States west of the 105th meridian, as shown in Fig. 1, is essentially mountainous; whereas much of the East and Gulf Coasts are bordered by a more or less gently sloping plain. The coastline in California, Oregon, and Washington in general is bold and rugged. Natural harbors are few and far between; most of them are found where drowned valleys exist. The sediments tend to be coarser than those in the harbors on the Atlantic Coast, perhaps owing to the greater relief and more arid climate on the West Coast. This inference, however, is subject to confirmation by detailed investigation of the problem.

San Diego Bay is essentially a drowned bay that has not filled completely since the glacial lowering of sea level. It is not evident as to whether the drowning is the result of post-glacial rise of sea level or whether it represents deformation or depression of the land. The sediments are sand and silty clay to a depth of at least 100 feet (Fig. 3).

San Francisco Harbor

San Francisco Bay has a complicated geologic setting. It has been one of the most active areas of deformation of the crust of the earth in the United States for millions of years. Its present form results from a period of crustal deformation prior to Mid-Pleistocene, during which time a series of hills and valleys comparable to those now seen in the City of San Francisco were formed (Louderback, 1953). At this time the Sacramento River flowed into the ocean through Golden Gate (Fig. 4). Subsequently, the area now occupied by San Francisco Bay was downwarped and the Berkeley Hills elevated During this elevation of the Berkeley Hills, the Sacramento River maintained itself through Carquinez Strait. The elevation of the hills and the downwarping of the bay went on at intervals during which several formations of sand, sandy clay, and clay were deposited. At least five formations have been recognized (Louderback, 1953); (Trask and Rolston, 1951). The oldest three

f these formations consist of alternating sequences of clay, sandy clay, and and. The upper two formations are sand and soft silty clay. They are as-ociated with valleys carved in the older and firmer sand and clay during late lacial and post-glacial time.

Recent work by the writer indicates that the lower three formations, which re older than the valleys formed during the last period of lowering of sea evel, contain a series of beds in definite order, beginning with waxy, fine-rained clay at the base, followed in turn by beds of sandy clay, and fine to edium-grained sand. Each bed in this sequence is 5 to 10 feet in thickness. his three-member sequence of beds is repeated at least five times, and has een traced in bore holes over a horizontal distance of more than five miles n the east side of the bay. Seemingly, these sequences are associated with limatic changes, which perhaps could represent various phases of glacial tages of the Pleistocene era. The waxy clays at the base of each sequence ive the impression of having been formed during relatively humid periods of limate. Sand grains in these waxy clays seem to have been dissolved and isintegrated during the process of weathering, whereas the sand in the over-ying sandy clays has not been so changed.

Following the deposition of these tripartite sequences of strata, the sea as lowered and a series of broad valleys, as much as 1000 feet in width, xtending to a depth of more than 200 feet, were cut in them. An interesting eature of this drainage system is its course south and west of Yerba Buena sland and over the rock sill between Yerba Buena Island and San Francisco, n which the present bridge stands (Fig. 4). This valley cutting of the late e age progressed in steps, as data from bore holes indicate the presence two terraces on the sides of the old valleys. The stream which drained e south end of the bay during early Pleistocene time went around the east d north sides of Yerba Buena Island. It did not extend across the sill, be-ause the sill at that time formed a hill at least 100 feet high.

The deposits on the surface of the bay consist of mud, sand, and gravel. he distribution of types of sediment on the present bay bottom is shown on ig. 5. This figure is based upon data presented by U. S. Coast and Geodetic rvey charts. Silty clay covers most of the bottom in shallow water and in aces protected from currents. On the east side of the bay this clay extends places as much as three miles from shore, in water less than ten feet deep. also fills the valleys carved in the older firm clays. In the areas of these cient valleys, the mud may be as much as 100 feet thick. On the San Fran-sco side of the bay, the clay likewise is thick in the sites of the old valleys. ypical bore-hole profiles are shown in Fig. 3.

The bottom of the bay in the center of Golden Gate is 360 feet deep and nsists of large blocks of rocks. On either side of the deepest part of the te, the texture of the deposits becomes progressively finer as the depth of ter lessens. First is a zone of fine gravel, which grades into coarse sand, th to the eastward and to the westward. Outside the Golden Gate is a escent-shaped bar, composed of fine to medium sand. This bar rises to thin 20 to 30 feet of the surface. It obviously represents an equilibrium ndition between tidal currents and sediment supply. Fine sand of about 0 microns diameter is found in the bottom of the channel northward through e upper bay and through the series of bays as far as the outlets of the San aquin and Sacramento rivers.

Portland Harbor

Portland Harbor is located at the junction of the Willamette and Columbia rivers, 100 miles above the mouth of the Columbia. A serious problem in connection with this harbor is the formation of bars in the Columbia River downstream from Portland. Most of the deposits brought down as bed load in the river are fine sand. The average diameter of this sand, according to Hodge (1934) is about 200 microns. Hodge states that sand drifts both to the north and the south at the mouth of the river, but the greatest drift is to the north.

Seattle Harbor

Seattle Harbor is located on Puget Sound. This sound represents a part of the earth's crust that has been sharply depressed in relatively recent geo logic time. The water is mostly 500 to 800 feet deep, with a maximum dept of 930 feet. The deposits consist mainly of soft, silty clay, with sand where sandy beaches are present. Puget Sound is connected with the ocean by Admiralty Inlet and the Strait of San Juan de Fuca. The water in Admiralty Inl is shallower than in Puget Sound, and in one place the inlet is crossed by a sill on which the water is only 240 feet deep. Puget Sound was covered by ic during the Pleistocene Age. Depth to bedrock varies greatly in the vicinity of Puget Sound, as shown by the borings illustrated in Fig. 3. The subsurfac deposits encountered in these bore holes consist of sand, silt, and gravel lef behind by the glacier, or by streams which reworked the glacial deposits. According to Fleming (1954), sea level was at least 250 feet lower in Puget Sound during the late glacial period than it is now.

SUMMARY AND CONCLUSIONS

The necessary requisites of harbors are protection from waves, sufficie depth of water, and space for maneuvering ships. These requirements are met in several ways geologically. The most common is inundation of river mouths by rising sea level, as represented by harbors at New York, Philadelphia, Norfolk, Charleston, and San Diego. Another process is subsidence and deformation of the land so that the sea occupies low areas. Examples are San Francisco, Seattle, and Boston. Another factor affecting harbor for mation is differential erosion of the land either before or after the sea cam in. Examples of differential erosion before the advent of the sea are Delaw and Chesapeake bays. Galveston Bay is an example of erosion by wave action after inundation. Bars deposited across the mouths of rivers, as at Galveston, also may lead to the formation of harbors. Deposits left by glaciers may form islands and leave deposits that protect bays from the open sea, as at Boston. Large and deep rivers, such as the Mississippi and Columbia, carry so much water that they keep their channels open to consider able depth for a long distance, thus allowing the development of inland ports such as New Orleans and Portland, Oregon. The depth of water in harbors is controlled by the amount of rise of sea level or sinking of the land and by the deposition of silt in the harbor. Factors that affect silting are distribution and strength of currents; quantity, size-distribution and mineral composition of sediment; amount and rate at which water is brought down by streams; distribution of salinity; type of clay minerals; and amount of orga matter and sewage in the water.

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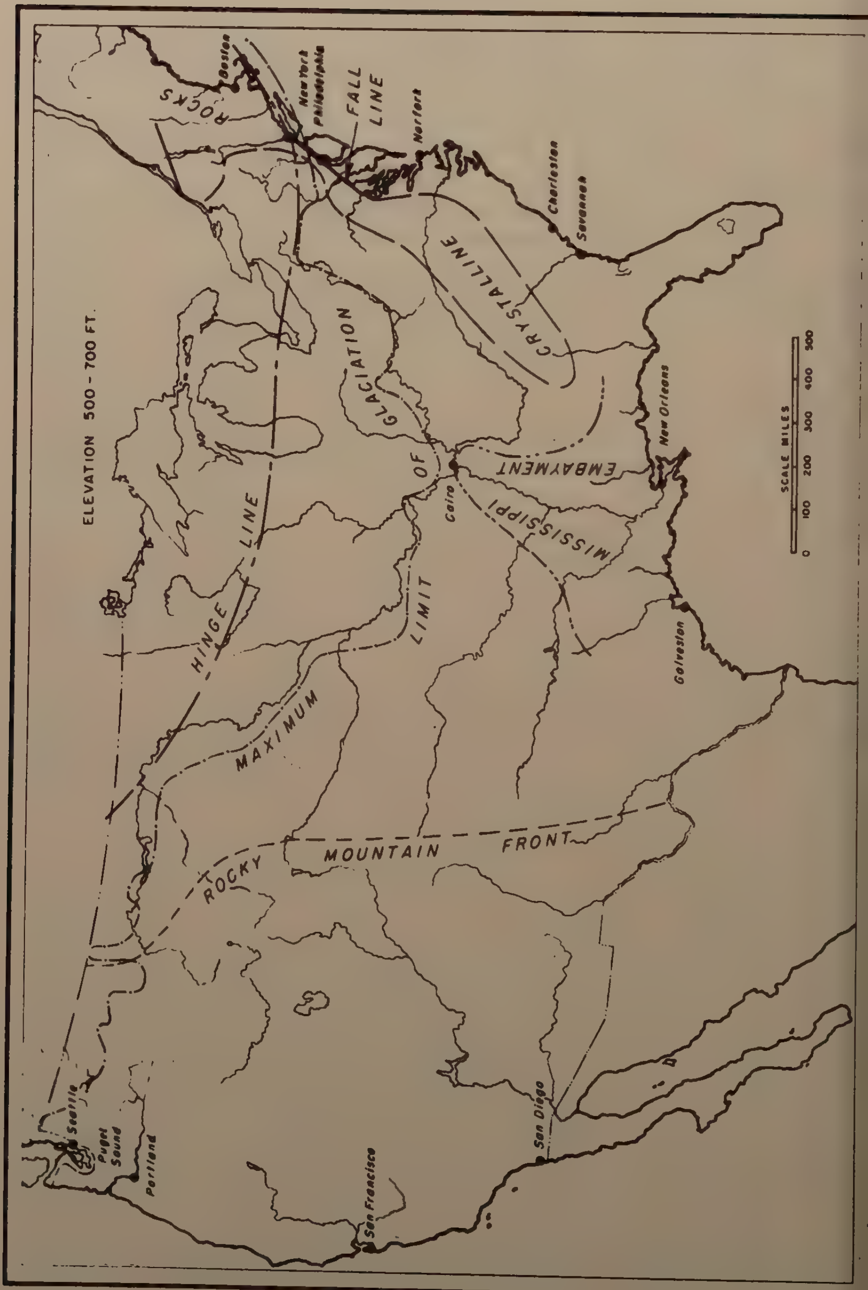
ig. 3. Logs of typical bore holes on the West Coast of the United States. From U. S. Corps of Engineers (1952).

ig. 4. Ancient valley patterns, San Francisco Bay, California.

ig. 5. Distribution of sediments, San Francisco Bay, California.


ELEVATION 500 - 700 FT.
ROCKS
Boston
New York
Philadelphia
FALL LINE
Norfolk
Charleston
Savannah
CRYSTALLINE
GLACIATION
OF
LIMIT
MAXIMUM
HINGE LINE
Cairo
EMBAYMENT
MISSISSIPPI
New Orleans
Galveston
SCALE MILES
0
100
200
300
400
500
ROCKY MOUNTAIN FRONT
Seattle
Puget Sound
Portland
San Francisco
San Diego

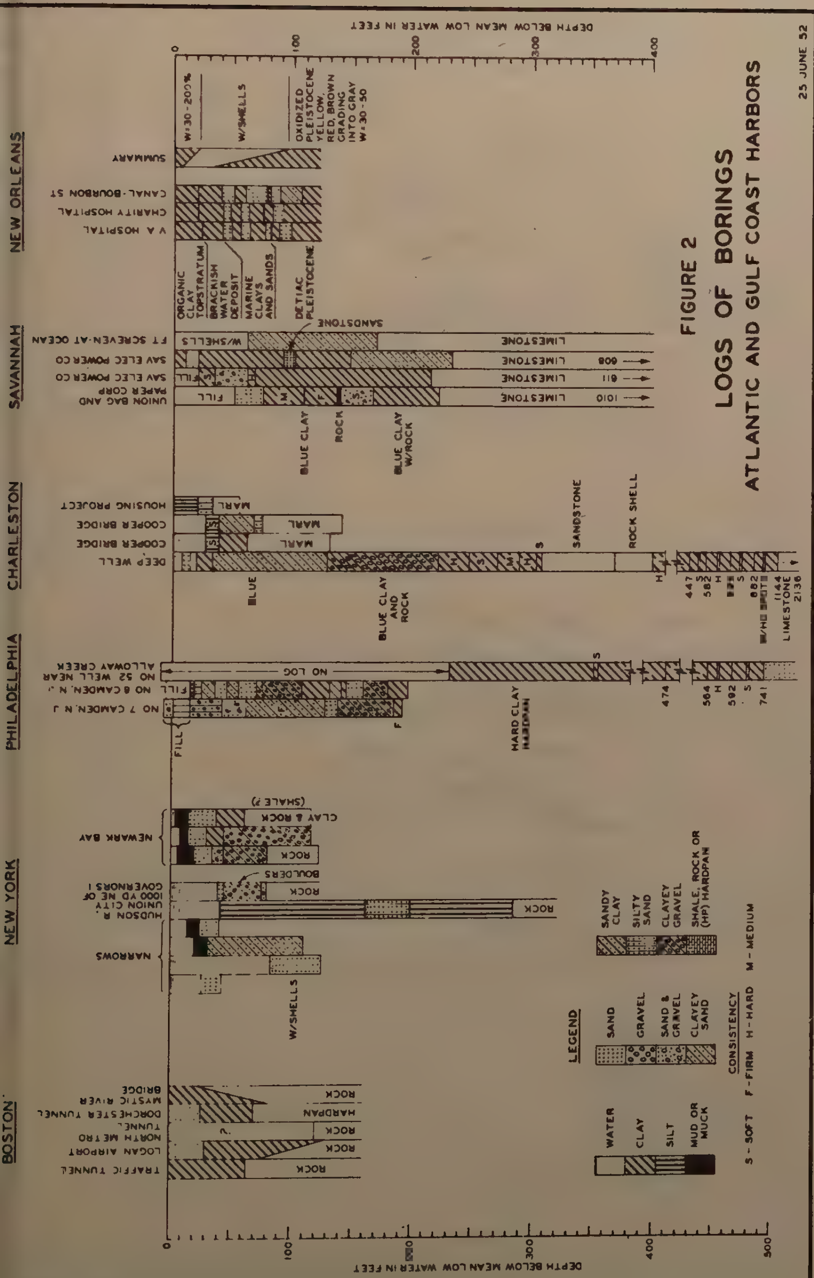


FIGURE 2

LOGS OF BORINGS

ATLANTIC AND GULF COAST HARBORS

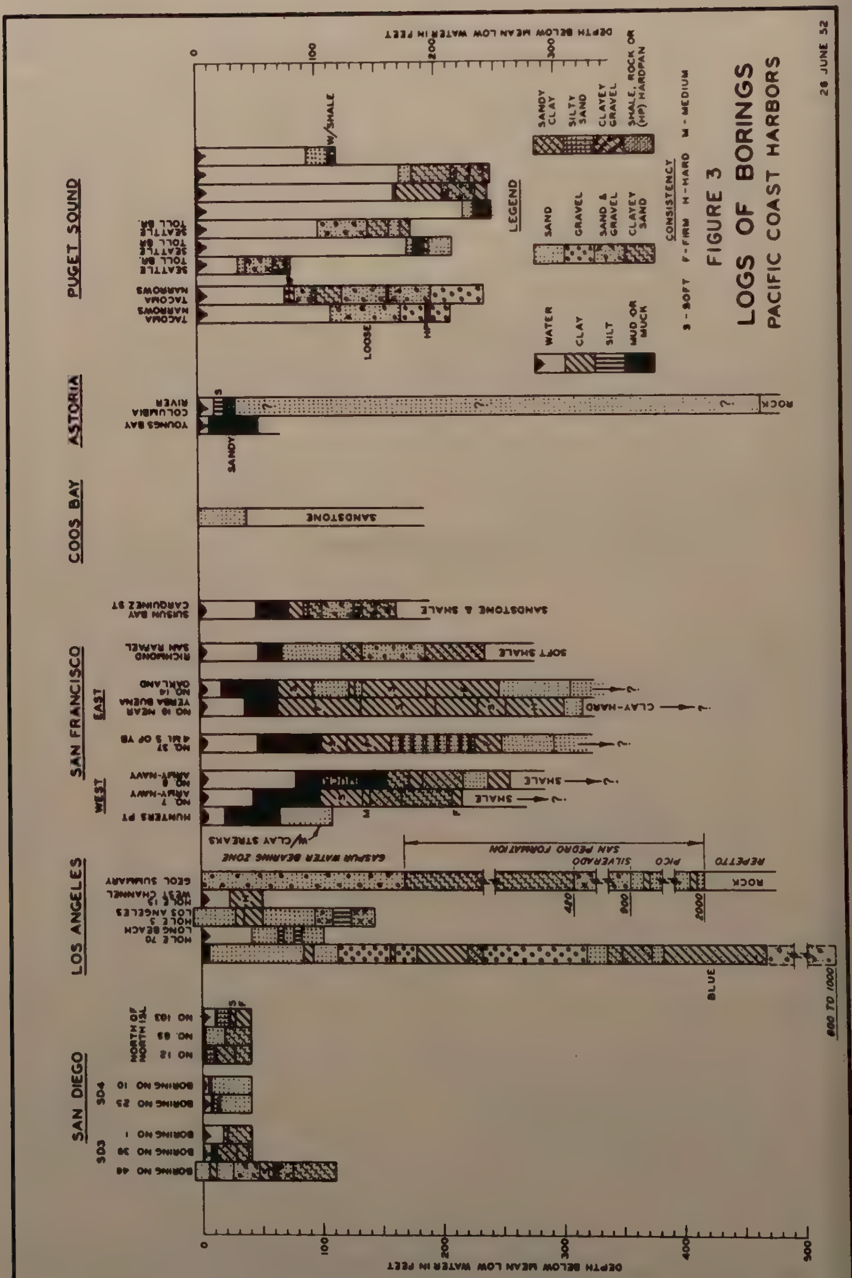


FIGURE 3
LOGS OF BORINGS
PACIFIC COAST HARBORS

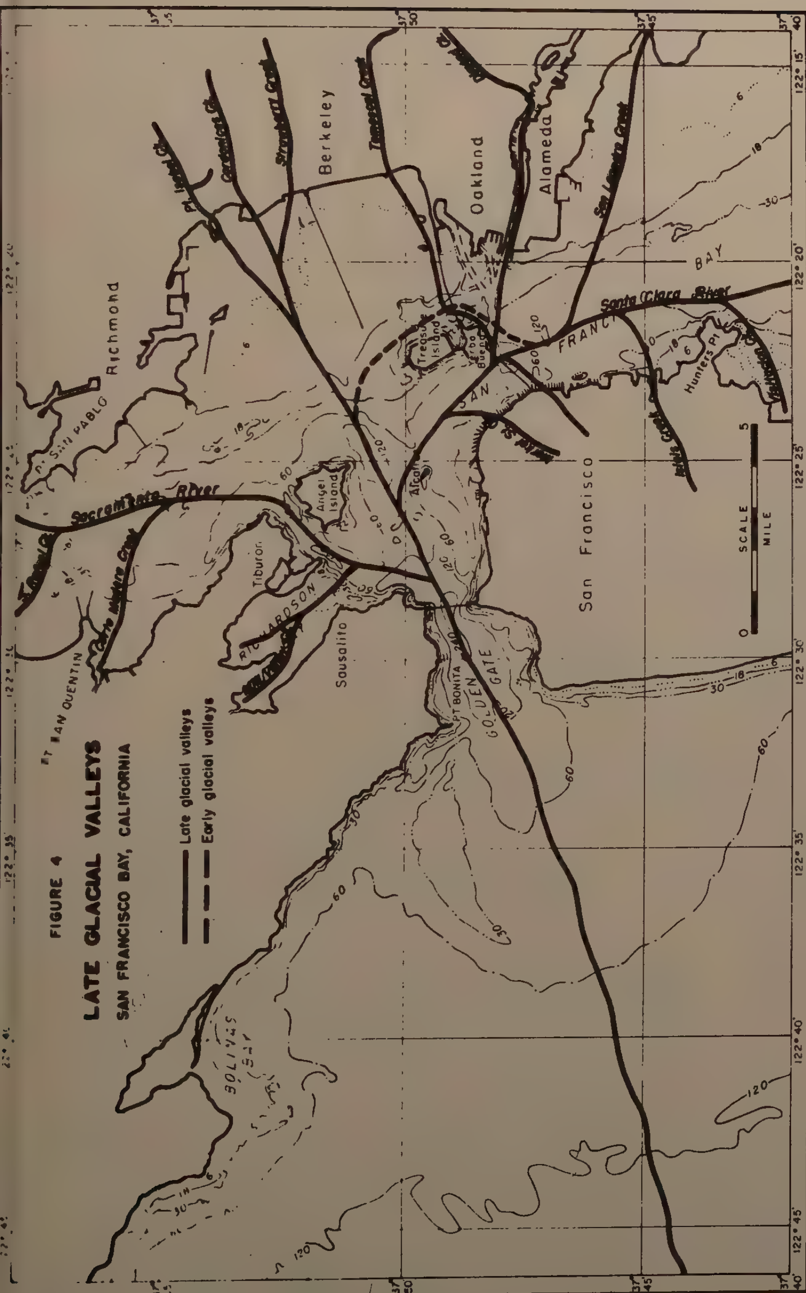


FIGURE 4
LATE GLACIAL VALLEYS
SAN FRANCISCO BAY, CALIFORNIA

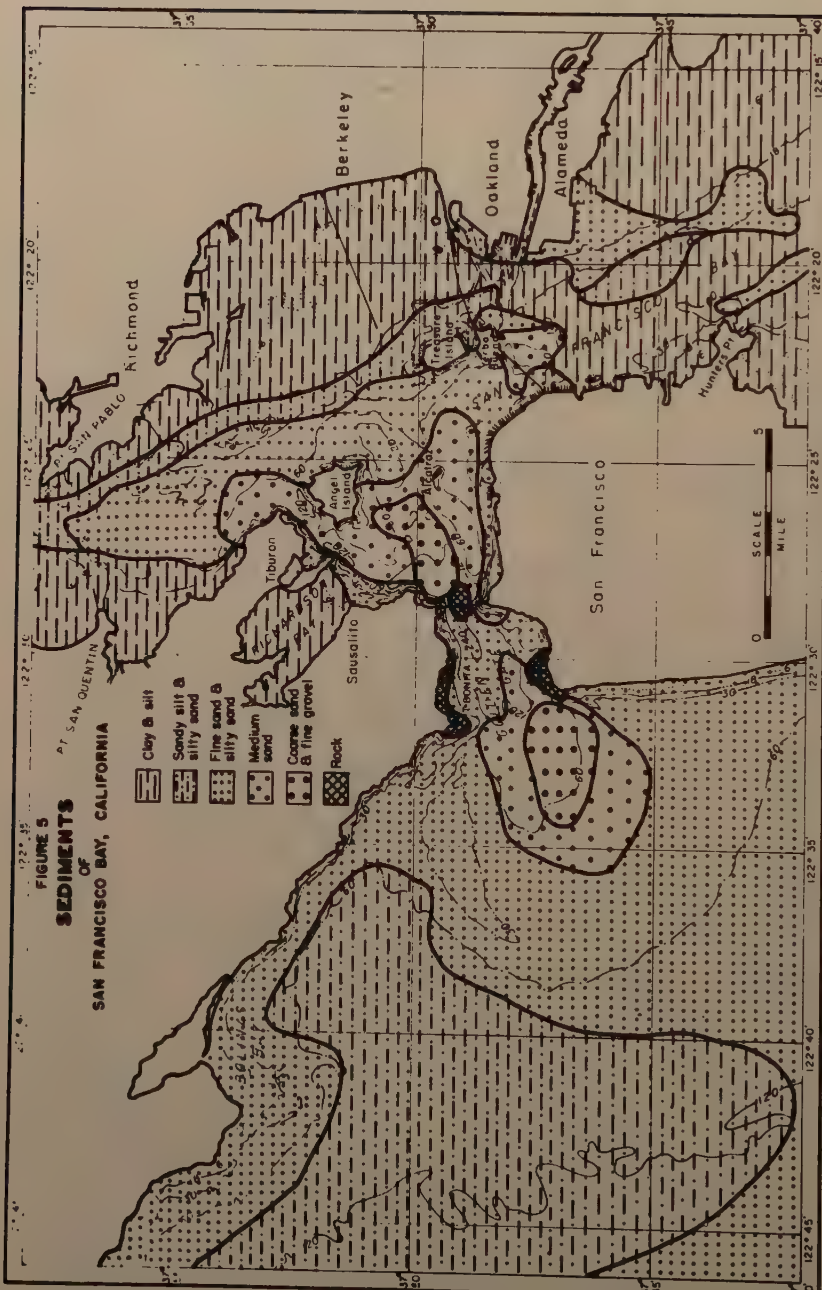


FIGURE 5
SEDIMENTS OF SAN FRANCISCO BAY, CALIFORNIA

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USE OF ZONING PRINCIPLES IN FLOOD PLAIN REGULATION*

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SYNOPSIS

Regulation of the use of flood plains should provide for protection of the floodway required for carrying flood waters and for the prevention of flood damage through unwise use of those parts of the flood plain not required for the floodway.

Zoning ordinances for the protection of the water-carrying capacity of floodways should prohibit all new construction or alterations of existing structures that may reduce the floodway cross-section or otherwise limit the capacity of the floodway. The construction of bridges, utilities, or other necessary structures in floodways should be permitted only after careful study by competent engineers has determined that the proposed structure will not unduly restrict the capacity of nor adversely affect the efficiency of the floodway.

The objective of regulation of occupance of the flood plain is to reduce or prevent damage resulting from flooding. This objective may be reached by the use of zoning ordinances, building ordinances, and subdivision control.

Good administration of zoning ordinances or other types of flood plain regulation requires that floodways and flood plains be clearly defined on maps so there can be no misunderstanding of the area subject to regulation. Elevations of flood heights or other controlling elevations should be stated in ordinances and such indefinite terms as "flood height" or "high water mark" should be avoided.

Quotations from several ordinances and laws, now in use in various cities, counties and states, furnish guidance for those who wish to draft ordinances for the regulation of flood plains in their own communities.

INTRODUCTION

The need for some regulation of the use of our flood plains to reduce the ever mounting toll of lives and damages taken by floods has been recognized

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for many years. However, little has been done to achieve any substantial degree of control of flood plain development until quite recently. Even now the efforts in this field are minute and scattered when compared with the country's overall flood control needs.

The vast damage resulting from the recent New England floods generated by hurricane Diane has given added emphasis and publicity to the need for flood plain regulation, particularly on the smaller streams where the control of floods by reservoirs is difficult.

The Engineering News-Record, on September 1, 1955, in editorializing on the effect of hurricane Diane on future flood control planning stated:

> "From a more optimistic viewpoint, - - there are things that can be done to minimize the destructiveness of floods in small river valleys if the affected communities will stand for the radical changes that are necessary and accept a reasonable share of the costs that are involved. For example, to provide a safe channel capacity, many buildings may have to be removed or set back, bridges raised and lengthened and the channel bottom cleaned of debris accumulated over many years. Possibly a parallel flood channel may be required, or flood walls or dikes may have to be built.
>
> "Outside the towns, pumping plants, power stations and factories that cannot be moved out of the flood plain can be diked. Inside or outside, dwellings may have to be prohibited in areas known to be subject to dangerous flooding. And finally, if sites for flood detention reservoirs are available, the necessary dams can be built."

The editorial points up two necessary actions of importance to planners. The redeveloping phase, in which the old mistakes of encroaching on the stream and its floodway are corrected, and the development phase, in which prevention of future damage is obtained by proper regulation of the use of the flood plain.

The corrective action needed in New England is no different from that needed in hundreds of other areas. In all such areas there is a need for comprehensive planning for flood control which will take into account all phases of community development.

Comprehensive Planning

The authority for local officials and planning commissions to develop and adopt master plans for the orderly development of governmental units and their environs is granted by state laws. The object of such legislation is to improve the health, safety, convenience and welfare of the citizens and to provide for future development of communities in a manner that will promote the general welfare and provide for the efficient and economical use of public funds.

The state planning laws contain provisions for master plans to include comprehensive surveys and studies of existing conditions and recommendations setting forth plans for the development, redevelopment, improvement, extension and revision of physical situations. Flood control and prevention are usually as prominent in the state enabling acts as sanitation, utilities, transportation, education and many other factors that are generally given prominent places in community plans.

Comprehensive planning for flood control at the local level should provide for coordination of all means of flood control that may be applicable to the local situation. Improved channels, local protection works, retention reservoirs, limited use of the flood plain, and structural adjustments to floods should all be considered and fitted into their proper place with respect to the master plan for community or area development. Planning commissions should make special studies of the flood problems with the help of competent consultants, and the resulting plan should coordinate both the development and redevelopment phases.

The master plan, however, is only an overall framework of desirable future developments and needs to be implemented by more detailed plans and regulations for accomplishment. Zoning ordinances, building ordinances and subdivision control are effective instruments for putting the master plan into operation.

Zoning is an extension of the police powers of the state to local officials, to be used as an integral part of planning to the end that public health, safety (from fire, flood and other danger), comfort, morals, convenience and general welfare may be promoted. Zoning ordinances must be reasonable and court decisions indicate that the factor of reasonableness, which the law requires, is strongly supported by a comprehensive, expertly-prepared master plan.

Floodway Requirements

In any plan of development the channel capacity requirements of the floodway must be given full recognition. This is essential to any sound plan that will permit the greatest use of the resources of the flood plain with the least damage.

At this point it might be helpful to future clarity to define the terms flood plain and floodway.

The flood plain is the area adjoining a river or stream which has been or hereafter may be covered by flood water.

A floodway includes the channel of a river or stream and those portions of the flood plains adjoining the channel which are reasonably required to efficiently carry and discharge the flood water or flood flow of the river or stream.

The capacity requirements of the floodway are determined by nature and cannot be ignored without disastrous results. Encroachments on a floodway by any property owner creates a nuisance that is detrimental to other property owners and the general welfare of the community.

Therefore, it becomes necessary to have two types of regulation of flood plains to prevent unwise land use. One serves to preserve and protect the floodway and the other to protect users of the flood plains. Both of these types apply in the development and redevelopment phases of planning.

Community Development Planning

The prevention of future flood damage resulting from occupance of the floodway or flood plain can best be accomplished through a master plan for community development which has given adequate consideration to flooding and to floodway requirements. Such a plan also will provide for adequate control of the floodway and flood plains.

Control of the Floodway

The objective of floodway control is to preserve adequate space for carrying flood waters without increasing flood heights and to limit land use which may unduly reduce the efficiency and capacity of the floodway.

The control or regulation, sometimes known as channel capacity regulation, as obtained through laws or ordinances that restrict or prohibit encroachments on streams by dumping waste along the banks, filling the parts of the flood plain required for the floodway, or construction of bridges or other structures which decrease the channel cross-sectional area or otherwise restrict the floodway.

This type of regulation is frequently contained in state flood control laws which are worded in general terms. For example, the Indiana Flood Control Act (1945) states:

> "It shall be unlawful to erect, use or maintain any structure in or on any floodway as a permanent abode or place of residence, or to erect, make, use or maintain any structure, obstruction, deposit, or excavation in or on any floodway - - -, which will adversely affect the efficiency of or unduly restrict the capacity of the floodway."

The states of Iowa, New Jersey, New York, Pennsylvania and others have somewhat similar laws to protect floodways.

Although state laws of the type quoted above are fairly effective in controlling the design and construction of bridges and location of large industrial plants, they are frequently inadequate in regulating the development of new subdivisions and controlling the filling of flood plains and dumping of waste along streams. The failure in this respect is largely administrative and is due to inadequate personnel and facilities for policing the state and to the lack of defined floodways throughout the state for the guidance of flood plain users.

The problem of encroachment on floodway and stream channels is one requiring constant vigilance and control at the local level. State and Federal agencies cannot and should not be expected to reach into every corner of the nation to police the actions of individual citizens who through ignorance or intent engage in objectionable encroachments.

At the local level, the city, county or regional development plan should provide a floodway zone or district which sets aside the area required for the floodway. All occupance of the floodway should be prohibited except for those uses which will not adversely affect its efficiency or restrict its capacity.

Some uses of floodways are definitely beneficial and may be permitted. Parkways and some agricultural uses, for example, will increase the efficiency of the floodway by keeping it cleared of undergrowth and other ob structions.

Necessary structures such as bridges, should be permitted only when designed and constructed in accordance with plans for the improvement of the efficiency and capacity of the floodway, or when adjustments are to be made in the floodway to compensate for any adverse effects of the construction. The determination of compliance with plans for floodway improvement or the adequacy of compensating adjustments should be made by competent hydraulic engineers.

Few local master plans and zoning ordinances are adequate in defining the limits of floodways and in providing for proper regulation or control. However, the zoning ordinance (1951) of Milwaukee County, Wisconsin, provides

an example of an ordinance in which a separate floodway district is established and regulated. The ordinance creates Upland, Valley and Channel Districts along streams, with the "upland" districts being on land above flood heights, the "valley" districts on the flood plains and the "channel" districts in the floodways. The ordinance for the "channel" districts contains the following provision:

> F-3—Channel District
>
> In the F-3 Channel District no building or structure, retaining wall or revetment wall, except bridges or dams, and no fillings --- or any kind of dump shall be placed. The boundaries of the F-3 Channel District shall be such lines as shall be officially determined from time to time by ordinance of the County Board ---.

The boundaries of a floodway district should be clearly and precisely defined on maps of sufficiently large scale that there will be no misunderstanding of their locations. The ordinance cannot be properly administered unless the location of the zone of prohibited structures is definitely known.

The determination of the floodway requirements and boundaries is a technical task that involves a study of past and probable future flood discharges, characteristics of the floodway, and possible modifications of the floodway. History of past floods is frequently insufficient to furnish a safe guide for floodway capacity requirements and future greater floods must be considered.

The technical experience for making such studies will be lacking on most planning boards and on most city engineering staffs. However, most states now have water resources agencies with responsibilities for making studies of floods and other water problems. Help in defining floodways usually can be obtained from such agencies. For detailed planning of improved channels or other works to increase floodway capacity it may be necessary to employ consultants with experience in this field.

In administering exceptions from the floodway zoning ordinance for necessary structures it may be advantageous for planning boards of small communities to require approval of state agencies as a prior condition to granting the exception, particularly if state laws prohibit encroachment on floodways and if adequate technical review of the project is given by the state agencies.

Control of the Flood Plain

The objective of control of the flood plain outside of the floodway should be the protection of the lives and property of the users. This protection may be obtained by preventing unwise use of the flood plain by limiting activities to those that will not be unduly damaged by flooding. The means of control are obtained through zoning ordinances, building ordinances and subdivision control.

Zoning ordinances employing land use regulations are a common devise for controlling flood plains. Many cities and counties now have such ordinances, of which the zoning ordinance (1953) of Kokomo, Indiana is an example:

> FP-1—Flood Plain District.
>
> This district is created to protect the public health and to reduce the financial burdens imposed on the community, its governmental units and its individuals by frequent and periodic floods and the overflow of lands. The boundaries of this district have been determined from data in the files

of the Indiana Flood Control and Water Resources Commission and as such lands become protected from the risk of overflow they shall be removed from the FP-1 District and be reclassified into the appropriate Use District in Section 13 of this Ordinance.

1. Uses Permitted:
   a. Any use permitted in an A-1 and R-1 zone as long as the use does not require the erection of a structure intended for residential purposes nor shall any existing building so located and used for residential purposes be enlarged or structurally altered.
   b. Public parks, playgrounds, and recreational areas so long as a structure intended for residential purposes is not erected.

2. Uses Prohibited:
   a. All uses, other than those permitted above are prohibited.

The permitted uses in the A-1 and R-1 districts mentioned in the Kokomo ordinance included general agricultural use, public parks, playgrounds, recreational areas, and buildings other than residences. Hence, the only prohibited buildings are residences.

Ordinances regulating only residences are too limited in scope and do not provide protection against the unwise use of flood plains by industries, utilities and others who may be subject to severe flood damage, and whose damage may have a greater general economic effect on a community than damages to residences.

The zoning ordinance (1949) of Azusa, California is somewhat broader in its restrictions than the Kokomo ordinance and has the following provisions:

Water Conservation Districts

Statement of Policy. There are certain areas of the City which under present conditions are not suited for permanent occupancy or residence by persons for the reason that they are subject to the periodical spreading of water and other hazards. Therefore, for the public interest, health, comfort, convenience, preservation of the public welfare, the City Council does hereby create districts within which it shall be unlawful to erect, construct, alter, or maintain structures of any type.

Use Prohibited. All buildings or structures, mining and/or removal of rock, sand, and gravel, or manufacturing any products from the same for commercial purposes.

The body of the Azusa ordinance contains a clear statement of the purpose of the ordinance and relates it to public safety and welfare, which is necessary for all such ordinances.

The Zoning Ordinance (1950) of Shelbyville, Indiana, is stated in somewhat shorter terms:

Article VIII—Conservancy Use
Section 13. C-1 Conservancy District
The following regulations shall apply in all C-1 districts.

A. Uses Permitted.
   1. Customary agricultural operations provided that no odor—or dust-producing substance or use shall be permitted within two hundred (200) feet of a residence or business district boundary.
   2. Municipal recreation uses.

3. No building or structure designed or intended for permanent use or occupancy other than fences will be permitted.

It should be noticed that the Kokomo ordinance termed the district a flood plain district and the other two ordinances used the terms conservation or conservancy. The Shelbyville ordinance contains no reference to flooding in any part of the ordinance although it was written to prevent unwise use of flood plains. The avoidance of reference to floods or flooding either in the title or body of a flood plain zoning ordinance defeats the primary purposes of protecting against flood damage by making the purpose of the act vague or even unknown and encourages relaxation in enforcement. Such practices are sometimes excused on the theory that favorable public relations cannot be maintained if a community's unfavorable or dangerous features are acknowledged. However, in a test, the court's attitude toward a flood plain zoning ordinance will be more favorable if the purpose of the ordinance is clearly stated and if public safety and welfare are stressed as a justification.

Building ordinances can sometimes be used as a means of reducing potential flood damage in flood plains. Such ordinances may require that the first floor of all buildings be placed above flood heights, may prohibit basements, or may require walls to be constructed to withstand water pressure.

Such ordinances may be a part of building code requirements or more commonly may be a part of flood plain zoning ordinances.

The Milwaukee County zoning ordinance (1951) makes use of structural adjustments in the portion of the ordinance creating "valley districts" (flood plain districts).

F-2—Valley District:

In the F-2 Valley District no building or structure will be erected, and no existing building or structure shall be moved unless the ground upon which said building or structure is to be erected and ten (10) feet beyond the limits of said building or structure shall, prior to or at the time of construction, be raised to such level that the main floor of said building or structure shall be not less than three (3) feet above the high water level, as shown on the maps above referred to. No basement floor or other floor shall be constructed below or at lower elevation than the main floor.

Prohibitions against building below the high water mark are to be found in zoning or building ordinances of other communities but frequently they are buried among other regulations and are not recognized as flood plain regulations. Also, the elevation of the high water mark to be used in each locality must be given in the ordinance or be shown on zoning maps, as in the Milwaukee County ordinance, if the ordinance is to be effective.

Subdivision control is a very effective way of regulating the development of flood plain areas because it controls the very beginning of urbanization. The control should begin with the approval of the plat plan and should continue through the development and construction stages. No construction should be permitted to start until adequate flood protection is provided for in the plan and made a part of the overall development.

The methods of control consist of restrictions on the use of flood plains for new subdivisions, requiring structural adjustments, or requiring the use of flood protection works. All of these methods have been used successfully.

The restrictions on use method is employed in the Kokomo, Indiana ordinance (1953) for the control of the subdivision of land:

1. No land shall be subdivided for residential use unless adequate access to the land over improved streets or thoroughfares exists or will be provided by the subdivider from said land to a thoroughfare maintained by the governmental unit; or if such land is considered by the Commission to be unsuitable for such use by reason of flooding or improper drainage, (underscoring added), objectionable earth and rock formation, topography or any other feature harmful to the health and safety of possible residents and the community as a whole.

The use of structural adjustments is employed by Hammond, Indiana in its subdivision control ordinance (1955) by establishing a minimum elevation for streets:

Part VIII—Minimum Street Elevation.

1. The minimum elevation of residential streets shall be 596 feet mean sea level.

2. Any special adaptation of this elevation so as to allow new streets to meet the grades of existing streets which are below this minimum shall be submitted in plan to the Board for written approval.

The control of residential buildings is obtained in another section of the Hammond ordinance which requires that "the site of any residential building shall be filled and/or graded so as to reach a minimum elevation of 1 foot above the top surface of the curb of the street facing the lot on which the site is located."

Flood protection works, such as levees, may be used to advantage in places and should be required by ordinance for urban areas if no other means of protection are provided.

No subdivision should be permitted to develop in an area subject to flooding unless adequate protection against floods is provided by the subdivider. If this policy is followed the subdivision will not become an economic burden on the community at a later date when enraged residents, after a flood, demand that flood protection be provided. Further, it will tend to prevent or retard the development of areas subject to flooding when other areas can be developed more economically.

The Hammond, Indiana zoning ordinance (1955) establishes Flood Plain Districts and permits construction of residences if protected by the owner. The Hammond ordinance reads in part:

Chart II—Flood Plain District

Uses Permitted

(c) Single family residences under the following conditions:

1. The owners of the land must provide and pay for a system of adequate flood protection works along the course of the Little Calumet river. This system of flood protection works --- must be submitted in plan to the Indiana Flood Control and Water Resources Commission and approved by that Commission --- before it is considered by City of Hammond authorities for action.

2. After approval of such plans by the Flood Control and Water Resources Commission, they shall be examined by the City Engineer and approved by the Board of Public Works and Safety.

4. The standards to be followed by the Board of Public Works and Safety for any dikes included in these works shall be as follows: Such dikes shall

be constructed of impervious material; minimum elevation 598 feet mean sea level; uppermost surface width ten (10) feet, graded level and paved with cement or slag capable of bearing vehicular traffic; slopes on sides 1-1/2 to 1, the river side being protected by an 18 inch dump of one to six inch stone rip-rap, the outside slope being closely seeded in grass.

5. The City of Hammond shall be given a permanent easement for such works and dikes and their rights-of-way for purposes of their maintenance and control.

The provisions of this ordinance are intended to either lift the burden of providing flood protection from the city and place it on the subdivider, or discourage the development of lands subject to flooding.

One subdivision at Speedway, a suburb of Indianapolis, Indiana, has been developed with a flood protection levee built by the subdivider. The impelling force that caused the subdivider to furnish the protection, in this case, was not the requirements of the city or county plan commissions nor flood plain ordinances, but the refusal of the Federal Housing Administration to guarantee loans on any of the houses to be built in the area until the flood protection works had been approved by the Indiana Flood Control and Water Resources Commission.

Redevelopment Planning

Unwise past use of floodways and flood plains has led to the growth of blighted areas in many communities. Such areas have a depressing effect on adjoining areas and are a deterent to wholesome community growth. The correction of such conditions calls for bold planning and a remolding of community activity. The corrective measures frequently extend over a long period of years.

The redevelopment should be a part of the community master plan and should include the removal from the flood plain of all activities that are seriously damaged by flooding, and the removal of all buildings and obstructions from the floodway. However, through the use of flood protection works and structural adjustments many desirable and wholesome uses may be provided for on the flood plain.

The master plan of the City of Cincinnati furnishes an excellent example of a river front redevelopment plan. The redevelopment area adjoins the central business section and has a most favorable location in relation to transportation and business. The plan calls for the removal of present decadent buildings and replacing them with parkways, stadium, heliport, administration and service center, convention and recreation center, and apartment buildings.

The stadium will be built to withstand flooding without serious damage, the heliport deck will be above flood levels and have parking space underneath. All buildings will be built on the higher ground in the area with first floors above flood heights. Floods will cause temporary disruptions of some uses, such as parking, but no serious damage will result.

SUMMARY

The floodway requirements of rivers and streams as well as the human occupancy of the flood plains must be considered if we are to obtain the

greatest use of the resources of our flood plains with a minimum of damage. It is of primary importance that adequate capacity be preserved in the floodway and that no encroachments be permitted. Flood plains, on the other hand, may be occupied under limited conditions.

Floodway ordinances should permit encroachment only for very limited purposes, such as bridges, and then only when structural adjustments are made to the conditions of the floodway and no adverse effects on the channel capacity of the floodway will result. Flood plain ordinances, on the other hand, may permit more general use but such use should be conditioned on the prevention of flood damage through the employment of non-damagable types of installations or activities, structural adjustments or protective works.

The limits of floodways should be based on thorough hydraulic studies of each local area. These limits, which will be determined by selection of a design flood for flood plain zoning purposes, will have a decided economic effect on the users of the flood plain and should be given careful consideration. Such studies of the magnitude and frequency of floods and channel capacity requirements are technical in nature and should be made by competent hydraulic engineers.

Many zoning ordinances place special emphasis on restrictions on residential use of flood plains. This possibly results from a mistaken idea that protection against danger to lives and damage to homes should be a principle consideration. This idea frequently has led to the illogical conclusion that areas unsuitable for residential purposes should be satisfactory for industry and should be so zoned. Protection for all community activities should be the primary consideration, including industry, transportation, utilities, and recreation. Flood damage to poorly located industries frequently is much greater and has a more far reaching effect on the economy of a community than does damage to residences.

The control of subdivision development for flood protection purposes should be at the point of approval of the plan for the subdivision. If the area to be developed is subject to flooding the original plan should not be approved until protection against flooding is provided. Once work has been started on the construction of streets and houses the problem of control becomes increasingly difficult as time goes on. Developers and other users of land in a flood plain frequently exert strong pressure against regulations that limit their use of land and such pressure will be intensified if restrictions are not enforced until after a large investment has been made in a partly completed improvement.

Good administration of zoning ordinances requires good tools and good inspection procedures. The tools are adequate zoning maps with floodway and flood plain districts clearly defined, pertinent elevations shown on maps or given in ordinances, and sufficient approved bench marks in each district for use in establishing correct elevations. Inspections should be made in the development area before, during and after construction to determine the degree of compliance with regulations and to obtain a basis for corrective action if required.

If planning commissions and zoning boards will accept comprehensive flood control planning implemented by flood plain zoning as being as necessary to the health and economic welfare of a community as planning for adequate streets, sewers, schools, shopping centers, industrial districts and other community improvements, then much can be done to limit and reduce the inconvenience, discomfort and damage resulting from floods.

DIVISION ACTIVITIES
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS BULLETIN-April, 1956

COMMITTEES 1955-1956

Executive
J. B. Tiffany, Jr., Chairman, W. M. Lansford, Vice Chairman
T. J. Corwin, Jr., C. E. Kindsvater, H. M. Martin Secretary.

Design
M. L. Dickinson, Chairman, L. J. Hooper, Don Mattern, J. H. Douma, Glenn Hands.

Hydrology
W. F. Guyton, Chairman, H. S. Riesbol, C. C. McDonald, W. E. Hiatt, D. K. Todd.

Floods
G. H. Jones, Chairman. R. D. Goodrich, H. A. Foster, J. I. Perry, Joseph Friedkin.

Research
J. W. Johnson, Chairman. J. S. Robertson, F. B. Campbell, D. R. F. Harleman, J. F. Ripken.

Sedimentation
Walter Moore, Chairman. H. V. Peterson, F. H. Larson, A. G. Anderson, A. P. Gildea.

Tidal Hydraulics
C. F. Wicker, Chairman. J. M. Caldwell, M. A. Mason, E. F. Fortson, L. P. Disney.

Publications
H. G. Dewey, Jr., Chairman. J. B. Tiffany, Jr., H. M. Martin.

Sessions Programs
J. B. Tiffany, Jr., Chairman

Task Forces

Design Committee
Manual on Gates and Valves for Reservoir Conduits.
Nomenclature for Hydraulic Design.
Energy Dissipators for Spillways and Outlet Works.
Friction Factors in Large Conduits.
Water Measurements.
Hydraulic Aspects of the Design and Operation of Spillway and Crest Gates.

Hydrology Committee
Review and summary of present policy on spillway design floods for dams.
Problems related to Hydrologic Data.

Task Forces (continued)

Flood Committee

Economic Effects of Floods on Agriculture.
Effect of Floods on Industry.
Effect of Floods on Transportation.
Maintenance and Operation of Flood Control Works.
Use and control of Flood Mains.

Research Committee

Open Channel Aerated Flow.
Cavitation in Hydraulic Structures.

Sedimentation Committee

Manual on Sedimentation
Rates of Reservoir Sedimentation
Distribution of Sedimentation in Reservoirs.

The Editor regrets that space limitations prevent listing the names of our members serving on the Task Forces. Please refer to the Official Register 1956 for their names.

The Tidal Hydraulics Committee has not appointed any Task Forces as yet since they are the newest addition to the Hydraulics Division. Chairman Wicker writes "Its area of interest is not merely the rise and fall of the tide and the flow in tidal waterways, but also the transportation and deposition of sediments under this complicated regimen, the effects of salinity intrusion on the tidal flows, and many allied subjects. At present, it is planning to broaden the interest in the subject and to expand the fund of knowledge by sponsoring papers at forthcoming Society Conventions."

Each year one member retires from a committee. Under normal conditions they have served your Division for five years. The success of a Division is in direct proportion to the amount of time and energy these men devote to the Society. Our Division is among the most active group, and it is our aim to maintain that position.

We will miss George Hickox, C. W. Thomas, G. R. Schneider, R. K. Linsley, F. B. Laverty, and A. S. Fry. There is a deep feeling of appreciation for their contributions to our Division.

Director L. E. Rydell has been appointed as Contact Member for the Hydraulic Division.

Glenn E. Hands, President, Kansas City Section reports that the Hydraulic Conference was a huge success, 347 attended with representatives from 17 states and 4 foreign countries. The papers were well received by the audience, reflecting careful preparation and presentation.

Harvey O. Banks, Acting State Engineer of California, is Chairman of the Task Force on Spillway Design Floods.

D. W. Van Tuyl is Chairman of the Task Force on Hydrologic Data.

William F. Guyton, Chairman, Hydrology Committee, plans to hold a meeting of the committee at an early date. He reports the two Task Forces are getting their work underway and it appears they will have a successful year.

The Committee on Snow, Ice and Permafrost originally a Joint committee was assigned to the Hydraulics Division. During the past two years your Executive Committee has questioned the advisability of sponsoring this group. Action was taken at the Berkeley meeting; the society committee on Technical Activities has been requested to transfer this group to the Soil Mechanics Division.

A. F. Ghiglione, Commissioner of Roads for Alaska, Chairman of this Committee, concurs with the action of your Executive Committee.

PLAN NOW TO ATTEND THE DIVISION MEETING

Location.
- University of Wisconsin at Madison, Wisconsin.

Date.
- August 22, 23, and 24, 1956.

Technical Sessions.
- Design
- Floods
- Hydrology
- Research
- Sedimentation

Host Section.
- Wisconsin.

Professor Arno Lenz, President of the Section and a member of our Division, is directing the planning.

Housing will be available at the dormitories for a package rate of $16.00 per person for Tuesday, Wednesday, and Thursday nights and 3 meals on Wednesday and Thursday, and 1 meal on Friday.

Tentative plans include an inspection trip to the Allis-Chalmers plant. A trip to a large brewery and also a night at the ball game.

Watch for further announcements in Civil Engineering. Details of accommodations at Hotels and Motels will be printed with the program.

Hydraulic Division Sessions at Society Conventions

To insure participation at all meetings, the Vice Chairman will be responsible for the programs for the next calendar year. By inaugurating this procedure there is a better distribution of the work within the Executive Committee.

The Technical Committees plan the programs often with papers of local interest as well as papers on subjects of general interest not only to Hydraulic Engineers, but to the entire profession. With three Society, one Division, and a number of Regional meetings each year these committees give generously of their time to insure the success of the meeting. It is necessary to plan ahead almost two years in order to give the author sufficient time to prepare his paper. The co-ordination of the program is difficult with this long range planning, since many engineers change jobs and may even move to other countries.

It is believed that some of our members have or could prepare papers that would qualify for publication if they understood the basic requirements. Presentation of a paper at a meeting is not a factor in determining its merits. Many contributions to engineering would not hold the attention of an audience because of the technical aspects, but would be of great value and should be published in the Transactions.

Papers of local interest will draw an excellent audience, but may fail to qualify for publication. In spite of the fact that authors obviously spend a great amount of time preparing their papers, we regret that it is not possible to publish all papers.

The following was prepared by the Chairman of the Publications Committee to assist authors in qualifying for a review of their paper by a board when submitted for publication.

Requirements Before Papers are Reviewed by Division Committee on Publications

1. Titles should not exceed a 50-character-and space length.

2. Type manuscript, original and two copies, double-spaced on one side of $8\frac{1}{2}''$ by $11''$ paper. Illustrations, tables, etc., must accompany each copy of the manuscript. Papers exceeding 18,000 words in length require special approval from the Society.

3. Forward the original and two copies to Mr. Harold T. Larsen, Manager, Technical Publications, ASCE, 33 West 39th Street, New York 18, New York. He will forward two copies to Mr. H. G. Dewey, Jr., Chairman, Hydraulics Division, Committee on Publications, c/o San Francisco District, Corps of Engineers, U. S. Army, 180 New Montgomery Street, San Francisco, California.

4. No line of mathematics should be longer than $6\frac{1}{2}$". Equations in the manuscript should be lettered in black India ink, with capital letters 3/16" high and all other symbols and characters in proportions dictated by standard drafting practice. Original typed equations are acceptable, but they will be reduced to 69% of their original size.

5. Tables must be typed (originals, not copies) on one side of $8\frac{1}{2}$" by 11" paper within an invisible frame $6\frac{1}{2}$" by $10\frac{1}{2}$". If tables are small, more than one should be included in the frame. Captions should be included within the frame. Reductions will be 69% of original size.

6. Illustrations should be drawn with black India ink on one side of $8\frac{1}{2}$" by 11" paper within an invisible frame $6\frac{1}{2}$" by $10\frac{1}{2}$". If some illustrations are small, more than one should be included in the frame. Captions must also be included within the frame. Glossy prints of photographs may be furnished when manuscript is first submitted. Reductions will be 69% of original size.

7. Submit with manuscript on a separate sheet, a 50-word abstract, typed double-spaced on one side of $8\frac{1}{2}$" by 11" paper.

8. Papers prepared for delivery on the program at technical meetings generally do not meet the above mechanical requirements. Accordingly, before submitting such papers to Mr. Larsen, appropriate revisions should be made. It is to be noted that delivering a paper at technical sessions does not imply such paper will necessarily be accepted later for publication in Proceedings. Each paper will be evaluated on its own merits by the Committee on Publications.

Presentation of Technical Papers

Chairman Tiffany sent the Editor a copy of a paper entitled "Toward Better Presentation of Technical Papers" published by the American Society for Testing Materials. With their permission we will quote from this article.

When you present a technical paper, do you interest and enlighten your audience, or do you irritate and stupify it? Unfortunately the Society experiences too many presentations of the latter type.**** After all the efforts by the author, the reviewers, the Committee, and the Editorial Staff, an unintelligible oral presentation is a grevious disappointment but one which is experienced only too often.

The relative merit of manuscript papers is not reflected in the way they are presented at the technical sessions of the Society. The best paper can be made a worthless mishmash by an author who fumbles his way through its presentation. On the other hand, the author of a paper which barely has

made its way through the reviewing screen can steal the show by a combination of alertness, stage presence, and detailed preparation.***

An author can do a great deal to improve his presentation without assistance. He can make a thoughtful selection of the major points in the manuscript and then arrange them in logical, outline order for speaking. These should be coordinated with visual aids, such as slides and charts.****

The author should exercise special care in the preparation of slides or other visual aids, and should make certain that such material can be seen easily from all parts of the audience.****

The Papers Committee is convinced that anyone who has the ability to prepare an acceptable technical paper also can present it effectively if only he takes the time to prepare his oral presentation.

Dead Lines For Papers

Title and Author	---6 months before meeting
Submit Paper	---4 months before meeting

This lead time is necessary in order to make local arrangements, have the program printed in Civil Engineering and have the paper reviewed by the Technical Committee arranging the session.

Papers of interest to Hydraulic Engineers often appear in the Journals of other Societies.

"Erosion Resistance of Concrete in Hydraulic Structures."
Journal of American Concrete Institute Nov. 1955

Forum on Hydraulics includes Water Hammer and Cavitation
Mechanical Engineering Dec. 1955

Membership in Two Divisions

In the January issue of Civil Engineering page 125 you will find the form for registration in a second division of your choice. It appears desirable to call this to the attention of your friends and associates if they are interested in joining two divisions.

The Board of Direction is to be congratulated in making this possible as many of our members have more than one interest in the field of Civil Engineering.

Editing of the News Letter

In past years the Secretary of our Division was also the editor of our News Letter. To spread the work among the Executive Committee members, it was decided that the Past Chairman would take this assignment. Since this bulletin's purpose is to keep you informed about our activities the Editor

would appreciate hearing from you as to what you would like included in this paper.

If you have any suggestions please write to the Editor. If you would like to become more active in Division work let us know about it. We can not promise to put everyone on a committee or task force on short notice, but a list of members that are willing to work would be of great value when planning for the future.

T. J. Corwin, Jr., Editor

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-sion sponsorship is indicated by an abbreviation at the end of each Paper Number, the bols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering hanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), tary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and ping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of ection are identified by the symbols (BD). For titles and order coupons, refer to the appro-te issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were ished in Journals of the various Technical Divisions. To locate papers in the Journals, the bols after the paper numbers are followed by a numeral designating the issue of a particular rnal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which cates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Founda-s Division.

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RIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

Y: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

NE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

LY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

GUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

PTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

TOBER: 809 (ST), 810 (HW)[c], 811 (ST), 812 (ST)[c], 813 (ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

VEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

CEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

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UARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 HW1)[c], 878(ST1)[c].

RUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 PO1)[c], 905(SA1)[c].

CH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

IL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 SA2), 959(PO2), 960(PO2).

iscussion of several papers, grouped by Divisions.

FILED IN STACKS

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